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PRELIMINARY INVESTIGATION OF INTEGRATED BLUFF PROTECTIVE SYSTEM--ETC(U)

JUN 79 W D REYNOLDS

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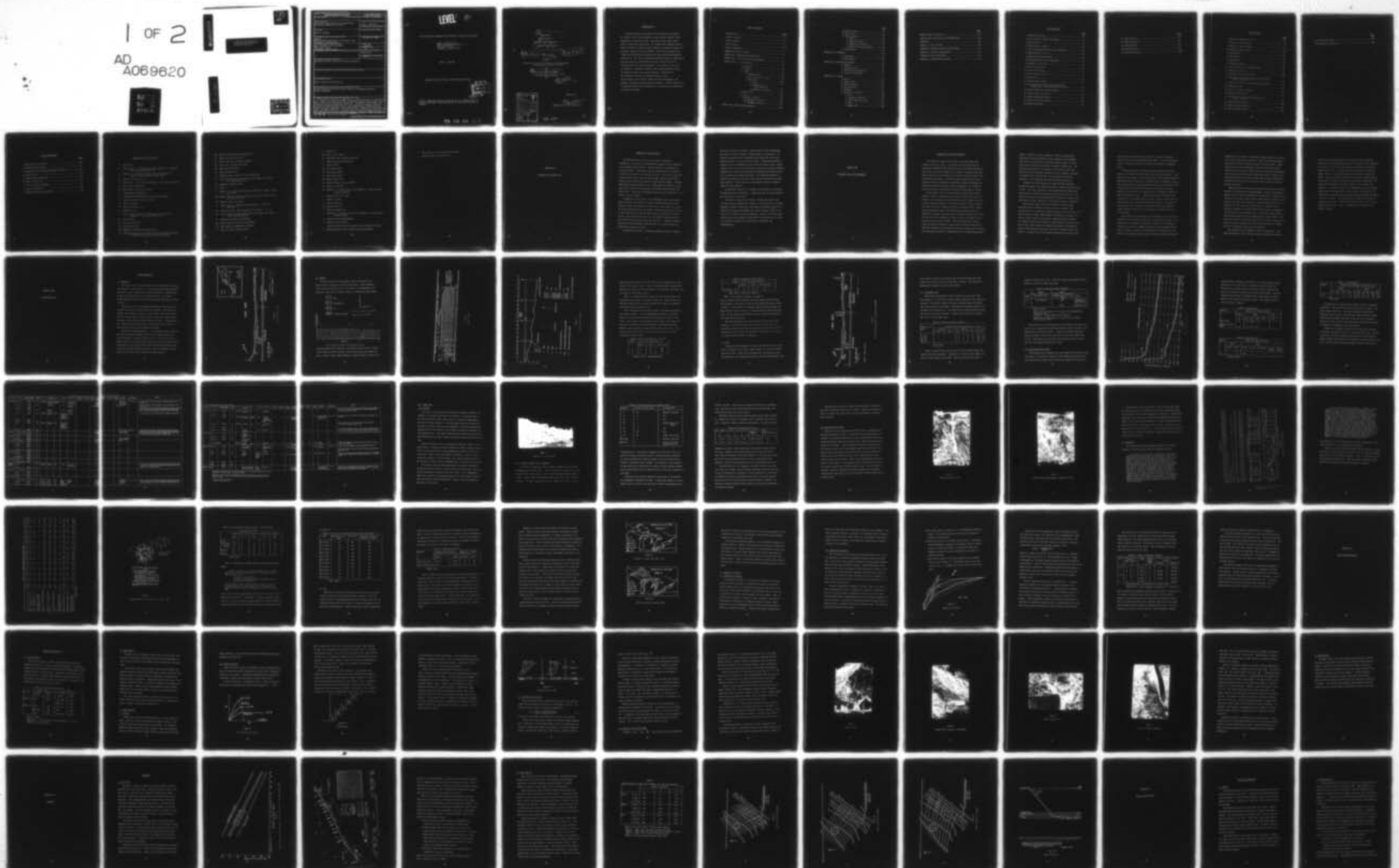

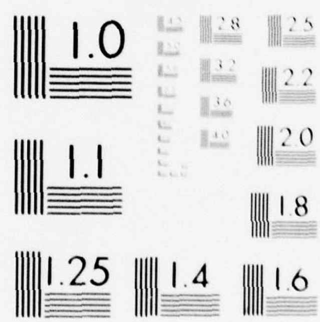

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The Lake Erie shoreline in Lake County is receding from one to five feet per year. There are no organized efforts to halt the recession and individual protection efforts have been mostly ineffective. The purpose of this thesis is to determine the physical mechanisms that cause recession, to develop a feasible preliminary integrated design system, and to develop design information using current analytical techniques. The conclusions are: Recession is caused by wave action and slope instability; individuals are not properly equipped to protect their own shoreline; and the integrated approach is feasible, though difficult to implement.

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INVESTIGATION OF INTEGRATED BLUFF PROTECTIVE SYSTEMS FOR LAKE ERIE

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Thesis, 1 JUN 1979

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NOMENCLATURE AND DEFINITIONS

A Area (ft^2);

b Chapter Three F. - Conjugate axis (mm); Appendix D - refracted wave orthogonal spacing (cm);

b_o Appendix A - Y intercept of regression line; Appendix D - deepwater wave orthogonal spacing (cm);

b_1 Slope of regression line;

c Chapter Four - Soil cohesion; Appendix D - wave celerity (fps);

db Wave breaking depth (ft);

ds Water depth at structure toe (ft);

F F probability distribution;

F* F probability distribution for two regression lines;

FI Slope of Atterberg Limits test line;

FS Factor of safety;

Gs Specific gravity of soil;

h Elevation (ft);

H Chapter Five, Appendix D - Significant wave height (ft); Appendix E - Design wave height (ft);

Ha Altitude (ft);

Hb Breaker height (ft);

H_B Bluff height (ft);

Ho Deepwater significant wave height (ft);

Ho' Deepwater wave height equivalent to observed shallow water wave if unaffected by refraction and friction (ft);

Hrs Refracted and shoaled wave height (ft);

k Runup correction factor (%);

$k\Delta$ Layer coefficient of rubble structure;

K_D Armor unit stability coefficient;

Kr Refraction coefficient;

Ks Shoaling Coefficient;

LI Liquidity index, measure of soil's plasticity;

LL Liquid limit, moisture content when soil changes from plastic to liquid behavior (%);

Lo Deepwater wavelength (ft);

m Slope;

n Appendix A - number of observations; Appendix E - number of armor rock layers;

NMC Natural moisture content, moisture found in unit volume of soil at a given time (%);

Nr Number of armor rock units;

p Chapter Three F. - Parallax (mm); Appendix E - armor rock porosity (%);

PI Plasticity index, range of plastic soil behavior = LL - PL;

PL Plastic limit, moisture content when soil changes from plastic to brittle behavior (%);

Q Longshore transport rate (yd^3/yr);

Qg Gross longshore transport rate (yd^3/yr);

Qn Net longshore transport rate (yd^3/yr);

r Armor rock layer thickness (ft);

R Runup (ft);

SSE Error sum of squares;

Su Undrained shear strength (tons/ft^2);

T Significant wave period (sec);

V Volume (ft^3);

Va Air Volume (ft^3);

Vs Soil volume (ft^3);

Vv Void volume (ft^3);

Vw Water volume (ft^3);

wr Specific weight of rock (lb/ft^3);

ws Weight of soil (lb);

ww Appendix A - weight of water (lb); Appendix E - specific weight of water (lb/ft^3);

W Weight of armor rock (lb);

x Number of blows;

x_e Easterly wind (%);

x_w Westerly wind (%);

Y Moisture content (%);

α Appendix A - significance level (%); Appendix D - refracted wave approach angle;

α_0 Deepwater wave approach angle;

γ Weight of soil (lb/ft^3);

ε Strain resulting from a stress put on a soil particle or mass;

θ Angle protective structure makes with the horizontal;

σ Stress put on a soil partical or mass; and

ϕ Internal angle of soil friction.

CHAPTER ONE

INTRODUCTION AND OBJECTIVES

INTRODUCTION AND OBJECTIVES

The Ohio shoreline of Lake Erie is used for industrial, residential, recreational, and agricultural purposes and is, therefore, a very valuable state resource. But the shoreline is receding at a considerable rate. Nine percent (9%) of the shoreline is receding at a critical rate and 37% at a non-critical rate. The critical recession involves significant erosion and major damage, whereas non-critical recession also involves significant erosion but minor damage. The Ohio Shore Damage Study by Bedford et al. (1978) shows $2.2 million residen-dential and industrial erosion damage from 1972 through 1976 for Lake County alone. Some type of protective system must be implemented if such losses are to be limited.

An objective of this thesis is to thoroughly study a Lake County reach, delimiting the problem of erosion by determining the physical mechanisms that cause recession in that reach. Lake County was selected for research because 23% of the state's critical shoreline erosion occurs in that county and 5.5 miles of the shoreline are subject to critical erosion, some protected and some not. The shoreline recession rate in the county varies from one to five feet per year. A reach of shoreline characterized by bluffs was selected because it is very difficult to defend bluffs against recession.

A consensus exists that an integrated protective system is the best

solution to shoreline recession. Such a system is often recommended but seldom, if ever, attempted. Another objective, therefore is to identify some preliminary integrated system designs for this reach, indicating what is and is not likely to work. A detailed engineering design will not be attempted but will remain for future study. Special attention will be given to the potential variability of design parameters throughout the reach, because extensive differences will make it difficult to implement an integrated system. The integrated approach is especially appropriate since the reach primarily consists of privately owned property. Public property is not normally extensive enough to apply this type approach.

An additional objective is to apply current analytical techniques in developing design information. Some design work was done previously, but old techniques were used.

The thesis is organized as follows: Chapter Two contains the literature review; Chapter Three contains a site description, including geography, geology, meteorology, and longshore transport; Chapter Four addressed the physical mechanisms that cause recession; Chapter Five consists of a lake level and wave analyses; Chapters Six through Eight contain design considerations, alternatives, and applications; and the last chapter is devoted to a discussion of conclusions and recommendations.

CHAPTER TWO

LITERATURE REVIEW AND BACKGROUND

LITERATURE REVIEW AND BACKGROUND

The problem of shoreline recession has not been extensively studied until recent times. The current interest stems from the tremendous development along shorelines and the resultant destruction when natural forces are allowed to act freely. The problem has been given attention at the federal, state, and local levels as well as by the University community. Studies have been aimed at defining the problem, gathering information, organizing to solve the problem, and determining methods/structures that are inexpensive yet effective. A brief overview of the studies performed by these agencies follows.

At the Federal level, the United States and the State of Ohio agreed in 1942 to a cooperative beach erosion study of the Lake Erie shoreline for the purpose of determining effective, economical methods of shore and beach protection for private and public shore frontages. The study, initiated in 1948 by the Corps of Engineers (COE), United States Army, and the State of Ohio Department of Public Works, was completed in segments, the Lake County segment being published in 1954. This was the first comprehensive study to address the problem of Lake Erie shoreline recession in Ohio. The COE concluded that cost per front foot would be reduced by protection of continuous stretches of shoreline regardless of structures used and that neither public interest nor organization existed for implementing such an integrated protective

system. Therefore, the COE suggested a number of alternatives, specifically beach nourishment, groins, bulkheads and revetments, that could be implemented by individual property owners or cities based upon their own determination of economic justification. The COE also concluded that, in accordance with Public Law 727, the recession of private property involved no public interest, therefore the United states could not share in the expense of shoreline protection. This compounds the problem of public motivation.

Although the United States still could not share shoreline protection expense, the Congress authorized a National Shoreline Study in 1968. The study, published in 1973, appraised shore erosion and shore protection needs for the entire United States shoreline. This study assisted the state by describing the shoreline in detail and by delineating critical, noncritical, and protected reaches of shoreline.

After the COE report (1954) was completed, the state began gathering general information necessary for the initiation and implementation of a shoreline protective program. The Ohio Department of Natural Resources (ODNR, 1959) gathered information on geology, lake levels, current projects and research, probable causes of erosion, and methods of controlling recession. Continuing to increase the data base, Pincus (1961) compiled an engineering geology of the shoreline which included geology, wind data, lake levels, littoral drifts, and profiles. No data had yet been gathered concerning the rate of shore-

line recession until general descriptions of reaches, including recession rates, were developed by ODNR (1961). All of these reports serve as information references and not as guidelines or suggested plans.

Once enough general information had been gathered, the state began to study specific hazard areas found along the shoreline. Since Lake County was subject to erosion and had a highly developed shoreline, ODNR hired Stanley Consultants, Cleveland, Ohio, to complete a study of the county shoreline from the Cuyahoga County line to the Chagrin River. Stanley (1969) summarized the mechanisms of shoreline recession, economic considerations, and suggested low cost remedial measures for the private property owner. Carter (1976) studied the entire Lake County shoreline and addressed the physical setting, erosion processes, and recession rates for each reach of the shoreline. The purpose of his study was not to recommend a protective system, but rather to compile pertinent data for use in developing a shoreline protective system design.

Some studies have been made on the mechanisms of bluff recession, although the predominant work has been on simple shore erosion. Bluff recession is more complex in that slope stability is as important a factor as wave action. Quigley and Gelinas (1976) developed some of the soil mechanics aspects of bluff recession, concluding that till bluffs are flattening to their angle of repose, retreating slowly but

maintaining their profile, or constantly changing profile as a result of toe erosion and slope instability. Bluff failure characteristics along Lake Erie were studied by Chieruzzi and Baker (1959). Their study emphasized slope stability in connection with wave action. They concluded that to halt bluff recession a beach must be developed, the toe must be protected, and a stable slope must be insured. The Wisconsin Geological and Natural History Survey (1977) addressed the predominant shoreline erosion and bluff stability mechanisms on the Wisconsin shoreline.

Numerous shoreline protection structures and installation methods have been developed and are being developed. The COE "Help Yourself" pamphlet and the Illinois Department of Transportation pamphlet entitled "Harmony With the Lake: Guide to Bluff Stabilization" present structural measures and installation guidelines. Demonstration programs such as Brater et al. (1977) study the effectiveness of structural bluff protection measures by actually building the structure, then watching its performance over a period of time. The COE has a demonstration program called Shoreline Erosion Demonstration Project (SEDP). Under provisions of SEDP, the COE (1977 and 1978) constructed offshore breakwaters of gabions, stapods, and Z-wall at Geneva State Park, Geneva, Ohio and is currently studying their performance.

The literature review suggests a number of conclusions. The federal government, though interested and concerned about shoreline

recession, cannot directly pay for or construct shore protection structures for privately owned properties. Protection must, therefore, be inexpensive and initiated at the local government or individual property owner level. The data that has been gathered, though not always complete, is appropriate towards developing an engineering design. Recession rates can and have been accurately determined, but the mechanics of bluff recession are still not well understood. The overlap of the hydraulics and soils fields has not improved this situation. The effectiveness of new, inexpensive structural devices, though currently being studied, is not assured. Additionally, the effectiveness cannot always be correctly determined because the problem is often ill defined. The accumulation of a beach is just that, an accumulation of beach. It does not necessarily mean a recession halt, though that conclusion was reached when the problem was not properly defined.

CHAPTER THREE

SITE DESCRIPTION

SITE DESCRIPTION

A. GEOGRAPHY

The subject reach is located about 20 miles northeast of the City of Cleveland's corporate limits. It is approximately 4,000 feet long and extends from a headland in the City of Willowick at 320th Street to the village boundary line at Eastlake/Lakeline. The reach is divided equally between the cities of Eastlake and Willowick as shown in Figure 1.

The shoreline runs in a generally northeastern direction and is characterized by 40 to 50 foot glacial till bluffs. The bluffs are steeper than 45 degrees except in a few locations where construction has cut the slope back to a milder angle. There are no major streams or harbors and no natural headlands within the reach. Although there are a few locations where the lake is at the bluff toe, a beach of small width is the more characteristic condition.

The reach is densely populated. It is privately owned except for the Eastlake sewage treatment plant, the end of 324th Street, and vacated properties lakeward of Shoreham Drive. Since the shoreline is highly developed, continued erosion would cause serious property damage and therefore, some property owners have armored their bluffs by constructing structural protective devices. Some are dumping fill and rubble over the bluff face in hopes of slowing recession.

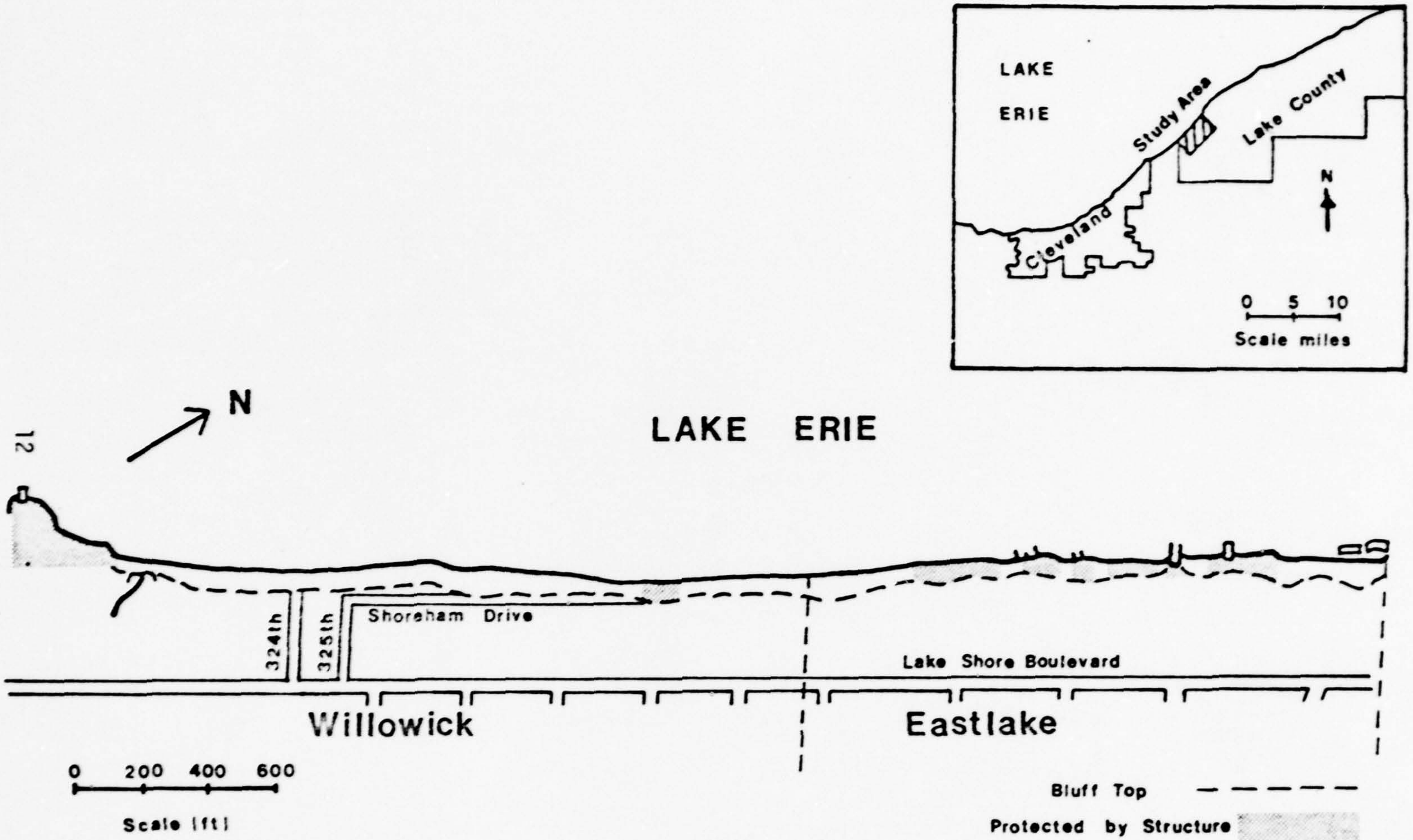


Figure 1

Shoreline Plan View

B. GEOLOGY

The reach consists of lake deposits from the Wisconsin era. Three principal deposits are found in the reach — shale (bedrock), till, and glaciolacustrine clay. The bluffs are predominantly till with small clay units as shown in Figure 2. Bedrock lies well below

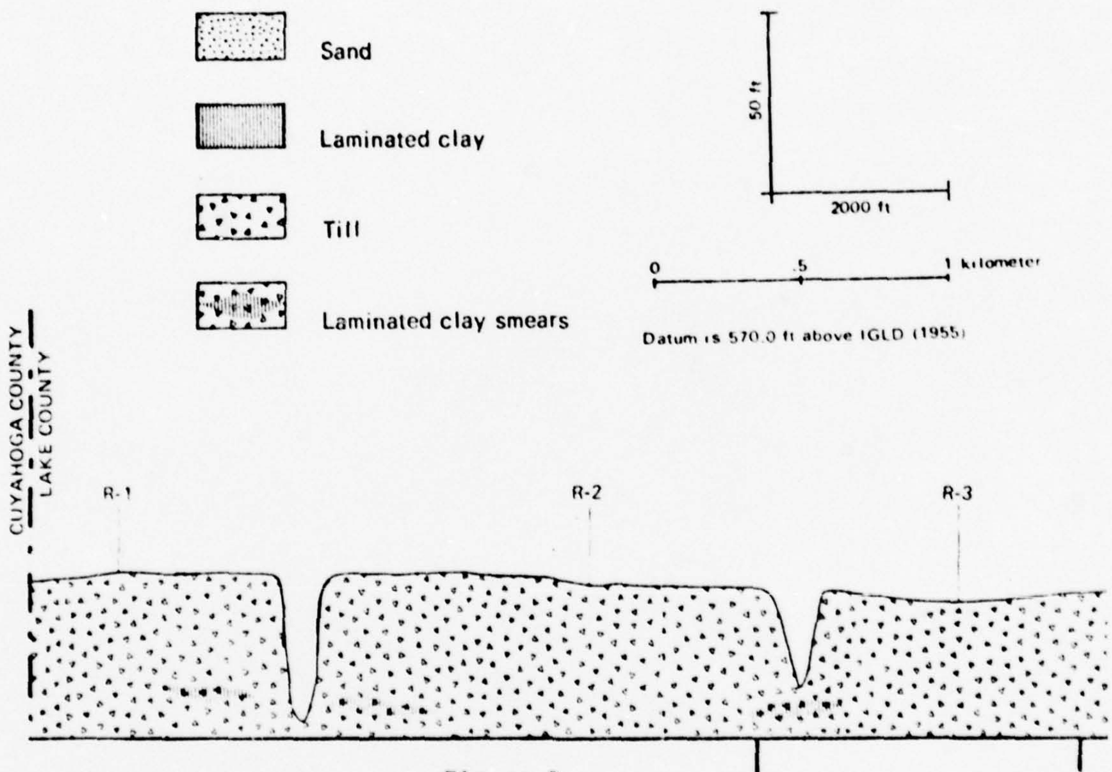


Figure 2

Cross Section of Shore Stratigraphy (Carter)

the lake bottom and is therefore not exposed to wave action. Figures 3 and 4 show the shale elevation found through borings and bottom samples. Carter's report (1976) contains a bedrock contour map that indi-

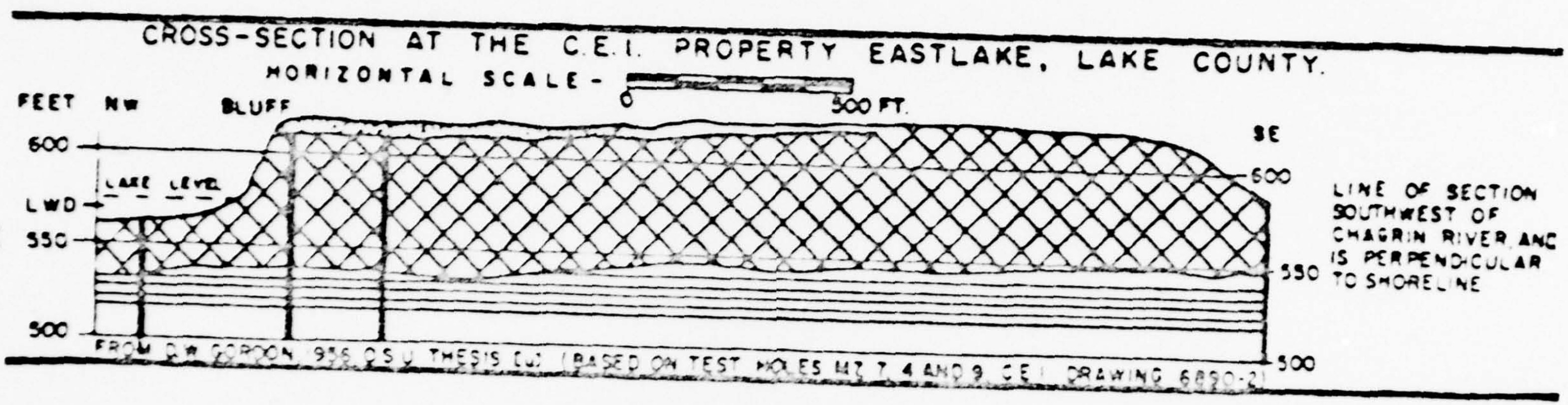


Figure 3.

Boring Profile

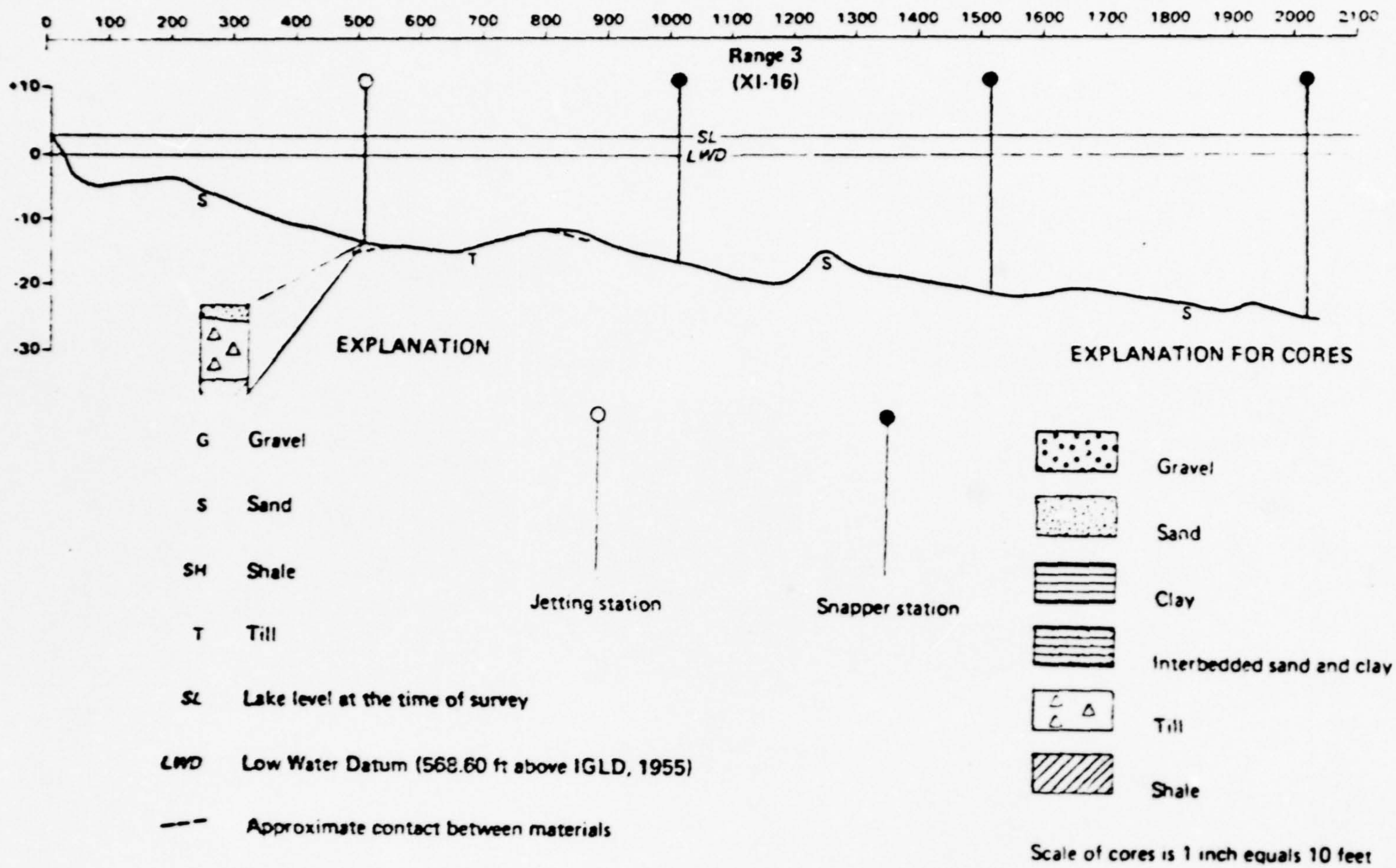


Figure 4

Bottom Profile Range 3

cates bedrock elevation at 550 feet (MSL) but that elevation is inconsistent with Figures 3 and 4. Since the bedrock contour map was extrapolated from a few pieces of known data, it is therefore not considered as accurate as the boring reports.

Shale is a sedimentary rock composed of clay size particles consolidated by the weight of the glaciers above it. Within this reach, the shale contains thin resistant siltstone beds and has a northeast strike with a gentle southeast dip.

Till is the material carried by a glacier and deposited out when the glacier retreats. It is a nonhomogeneous mixture of different grain sizes and different materials that overlies the shale in this reach. It is exposed to wave action at lake level. This tough, compact till is blue-gray in color when fresh, and yellow-brown when weathered and, according to Carter (1976), joints are common. In general, the till contains 80% silt and clay, 15% sand, and 5% gravel according to Tables 1 and 2.

Table 1. Grain-Sized Analysis of Till.[1]

Range	Grain Size (Weight Percent)		
	Gravel	Sand	Silt & Clay
2	5	15	80
3	4	13	83

[1]Carter (1976) - Combined Analysis

Table 2. Analysis of Bluff Sample[2]

Range	Grain Size (Weight Percent)		
	Gravel	Sand	Silt & Clay
Vicinity E. 332	6	31	63

[2]COE (1954) - Mechanical Analysis

NOTE: For location of ranges, see Figure 5.

The test results differ slightly, but would compare better had the COE performed a wet analysis as well as the mechanical analysis shown in Table 2. Since there is a measurable portion of grains above and below the #200 sieve in Table 2, a combined analysis should have been done. The wet analysis would have indicated a greater percentage of silt and clay and less sand.

The glaciolacustrine clay, gray to brown in color and thinly laminated, is the fine material that settled out of glacial lakes during the periods of freezing when sediment could not enter the lake. According to Carter (1976), the principal clay mineral in this reach is illitewith minor amounts of chlorite.

C. ZONES

To facilitate description of the Lake County shore, and to allow direct correlation with Carter's report (1976), the reach is divided into three zones: shore, beach and shoreline, and nearshore zones. The shore zone lies landward of the beach; the beach and shoreline zone

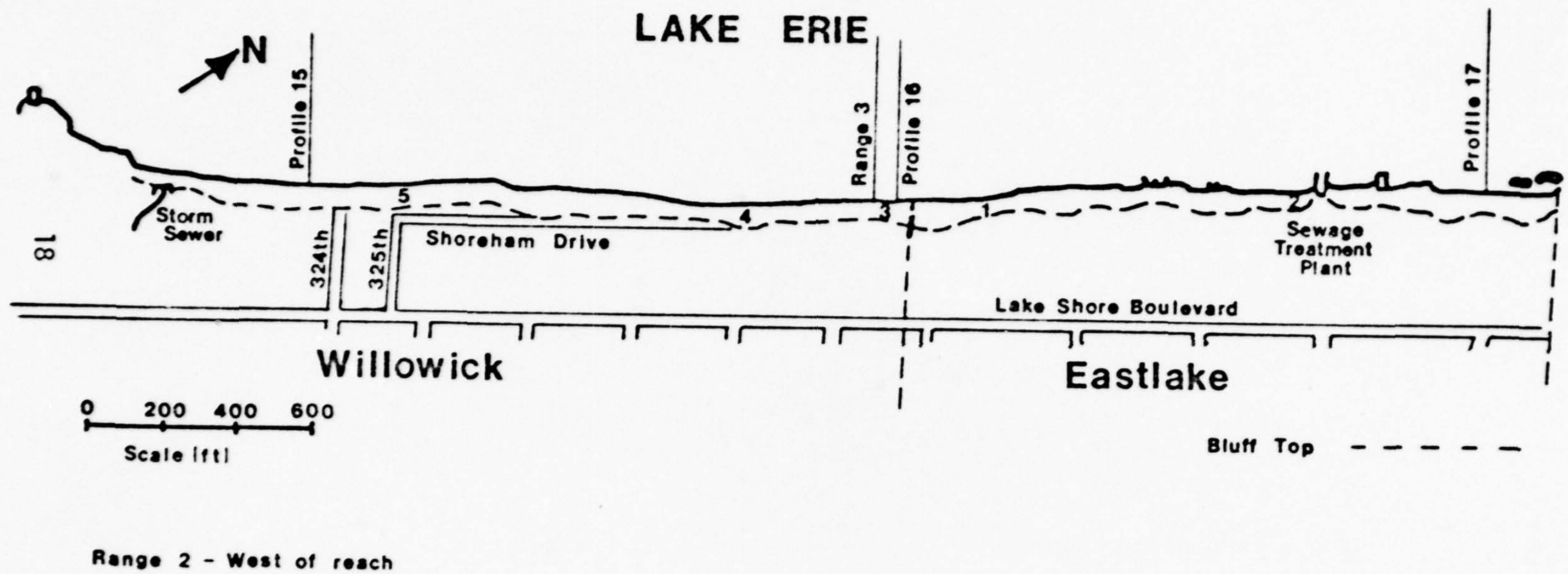


Figure 5

Soil Testing Locations, Profiles, and Ranges

lies between the shore zone and the lake; and the nearshore zone lies between the shoreline and about 2,000 feet offshore. The description and physical composition of each zone follows.

C.1. NEARSHORE ZONE

Sand extends continuously from the shoreline to more than 1,800 feet offshore. The sand forms ripples, dunes, and/or bars and, according to Table 3, COE (1954), grades from coarse to medium sand at the beach into fine sand and silt offshore. Sand thickness decreases offshore from the beach and shoreline zone. Till underlies the sand and breaks through the sand in some places offshore. Figure 4 shows where shale underlies the till in the nearshore zone.

Table 3. Analyses of Bottom Samples.[1]

Profile #16 Depth	Cumulative Percentage Retained by Sieve										
	Gravel					Sand					Silt
	1"	3/4"	1/2"	3/8"	#4	#10	#20	#60	#140	#200	#270
6 feet	-	-	-	-	-	0	1	37	92	94	95
18 feet	-	-	-	-	-	0	0	3	61	89	96
30 feet	-	-	-	-	-	-	0	0	7	18	27

[1]COE (1954)

Table 4, Carter (1976), indicates that the lake bottom slopes downward at less than one degree. The profiles in Figure 6, COE (1954), bracket the profile found in the reach. They show that the mild slope is

consistent throught the reach. There are no major protruding structures within the reach that affect the slope.

Table 4. Nearshore Slopes and Bars[2]

Range	Slope[3] (Water Depth/Distance From Shoreline)	Bars[4] Distance from Shoreline To Approximate Bar Crest (Feet)	Shape
#3	28 ft/2000 ft	800	Moderately Well-Defined
		1250	Well-Defined

[2]Carter (1976)

[3]Lake level was between 2.7 ft and 3.1 ft above LWD when survey was done.
A slope of 1 degree equals 18 ft/1,000 ft.

[4]Moderately well-defined, 3-4 ft relief; well-defined, 5 ft or more relief.

Table 4 shows that two nearshore bars are common to the reach with one bar occurring approximately 800 feet offshore, while the other occurs approximately 1,250 feet offshore These bars indicate that waves break twice before reaching the shoreline and that the offshore slope is mild. The actual size, location, and migration patterns of these bars were not determined during this study.

C.2. BEACH AND SHORELINE ZONE

Sand beaches occur throughout the reach except at the west end where the seawall is located well lakeward of the neighboring beaches and at the locations where rubble and fill have been dumped out into the lake.

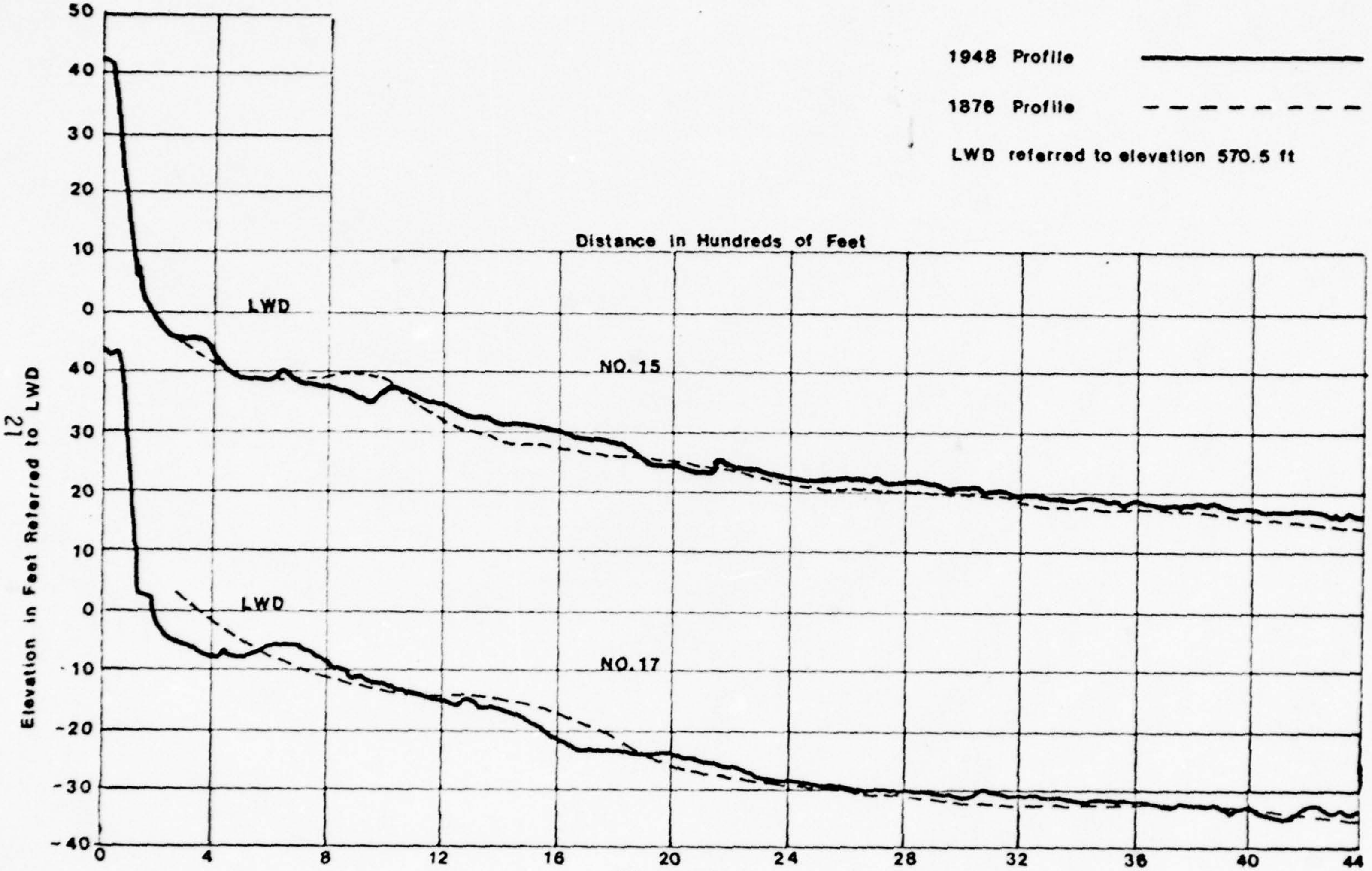


Figure 6

Bottom Profiles, 15 & 17

The beach width, dependent on lake level, is wide at low lake levels and narrow at high lake levels. According to Carter (1976), the beaches have an average vertical thickness of about 5 feet and are generally composed of medium to very coarse grained sands. Two sand analyses were performed in November 1978 and they appear in Table 5. Tables 6 and 7 summarize previous analyses. The locations for all tests are found on Figure 5.

Table 5. Sand Analyses

Location	Grain Size (Weight Percent)					
	Gravel	Sand				Silt
	#4	#20	#40	#100	#200	Pan
Storm Sewer	13	37.1	46.8	3.2	-	-
Sewage Treatment Plant	0.3	11.3	77.2	11.2	-	-

Table 6. Sand Analysis[1]

Range	Grain Size (Weight Percent)						
	Gravel		Sand				
	4-16mm	2-4mm	1-2mm	0.5-1mm	0.25-0.5mm	0.125-0.25mm	0.062-0.125mm
3	-	7	86	1	-	-	-

[1]Carter (1976)

Table 7. Sand Analysis[2]

Profile	Cumulative Percentage Retained on Sieve									
	Gravel					Sand				
	1"	3/4"	1/2"	3/8"	#4	#10	#20	#60	#140	#200
16	-	-	7	12	29	33	36	96	99	100

[2]COE (1954)

The shoreline is slightly convex overall and has a 230 degree orientation. The reach plan view is fairly regular in shape. The major irregularities are: the seawall-protected headland that protrudes into the lake at the west end; concave portions of unprotected property between structures at the east end; and convex portions of rubble and fill that protrude into the lake when dumping is especially concentrated at any one property.

Table 8 has a complete listing of all the existing structures in the beach and shoreline zone, to include the shore zone. The only significant protruding structures are the jetties at the Eastlake Sewage Treatment Plant which have slight beach accretion on their west side. The four groins in the reach appear to hold no beach at all.

Table 8 Existing Structures (West to East)

Location	Plat Number	Shoreline Damage Survey 1977 No.	Frontage Feet	Permit #	Current Structure[2]	Built	Condition	Composition	Elevation-ft above LWD Shore	Lake	Width/ Length	Prev Struc
E. 320 St.	44-D-8,9	71M0669	125	973477035	Bulkhead & Boathouse	1969-imp 1973	Good	Concrete-4 Shore returns				Concr Stone Groin Break
	44-D-11	71M0668	220		Bulkhead		Good	Reinforced Concrete			15' wide	
	45-J-1,2	71M0667										
	45-J-3,4	71M0666	200		Sewer Outfall		Fair	Concrete Culvert				
	45-J-5	71M0665	100	973477079	Bulkhead Fill	1973	Partially Covered by Debis Pushed Over Bluff Top	Concrete				
	45-J-6	71M0664	80									
	45-J-7	71M0663	100	973477023	Bulkhead Fill	1973	Not Found-Covered by Debris Pushed over Bluff Top					
	45-J-10	71M0662	100									
E. 324 St.	City of Willowick (Street)		40									Groin (3 ea) Block
	45a-1,2,3	71M0661[1]	110									
E. 325 St.	45-d-81	71M0660[1]	50									
	45-d-82	71M0659[1]	50									Groin in vi 655)
E. 326 St.	45-d-83,84,85	71M0658[1]	150									
	45-d-86	71M0657[1]	50									
	45-d-87	71M0656[1]	50									
	45-d-88	71M0655[1]	50									
E. 327 St.	45-d-89	71M0654[1]	50									
	45-d-90	71M0653[1]	50									
	45-d-91,92	71M0652[1]	100									
E. 328 st.	45-d-93	71M0651[1]										
	45e-87,88,89,90	71M0650[1]	200									
E.329 St (Range 3)	45e-79		60	977470003	Bulkhead	1977	Good	Concrete-2 Shore returns				
	45e-79	71M0649	340									
E. 330 St.	City of Willowick (Street)		40									
	45e-91	71M0648[1]	60									
	45e-92	71M0647[1]	65									
E. 331 St.	10-I-63	71I0646										
	The Shore Club of Willow Park		375	975477013	Seawall	1976	Some Toppled By Fill Pushed Over Bluff Top	Concrete Filled Drums				Stone Groin Bulkh (12 e
	10-I-64	71I0645	60	973477067	Seawall	1973						
	10-I-65	71I0644	60	973477068	Seawall	1973						

ing Structures (West to East)

Elevation-ft above LWD Shore	Lake	Width/ Length	Previous Structures	Previous Identification[3]	Remarks
		15' wide	Concrete, Stone Block Groin (7 ea) Breakwater	#25, XI-67	Headland is being flanked on West end. Some concrete blocks still in nearshore zone. Continuous bulkhead from 669 through 667. East shore return as far as sewer outfall. Outfall was inshore-recession brought it within 50' shoreline. There is a pool from outfall to actual shoreline. Ownership may be Willowick-no permit from COE because originally construct inland. (Prior to 1968) Concrete slabs are low to the water—more a revetment than bulkhead.
			Groin Field (3 ea) Concrete Block Seawall	#26,XI-68	Bulkhead on 663 was probably covered over by fill/debris that has been dumped over the bluff top in numerous places along this stretch. No sign of previous structures visible. Shoreham Drive right-of-way has been encroached upon and property lakeward is gone.
			Groin (somewhere in vicinity of 655)		
					5' shore returns. Slump was caused by broken water main seeping water into bluff. Fill being placed to re-establish Shoreham Drive. Failure pending due to increased overburden.
			Stone Block Groinfield Bulkhead (12 ea)	#27, XI-69	Continuous seawall from Shore Club through 644. Appears to be no shore returns. Landward fill has toppled some drums and pushed others lakeward. No toe protection, filter, or support was placed under drums.

2

Table 8. Existing Structures (West to East) (Con't)

Location	Plat Number	Shoreline Damage Survey 1977 No.	Frontage Feet	Permit #	Current Structure[2]	Built	Condition	Composition	Elevation-ft above LWD Shore	Lake	Width/ Length	Previo Structu
E. 332 St.	10-I-66	71I0643	60	973477054	Bulkhead With Fill	1973	Fair	Steel Sheet Piling				
	10-I-67	71I0642	60									
	10-I-68	71I0641	60	973477083	Bulkhead— 3 groins	Improved 1973	Good	Bulkhead- Concrete w/sheet pile. Groin-concrete Hook	3.5	3.0	5/30	Concret Seawall field (
	10-I-69	71I0640	60	973477122	Bulkhead	1973	Good	Steel Sheet Pile				
	10-I-70	71I0639	60									
	10-I-71	71I0638	60		Seawall, 11/2 groin		Fair	Concrete				
E. 334 St.	10K-15	71I0637	60		Concrete Blocks On Nearshore							
	10K-14,13	71I0636	125		Slope							
	10K-11,12	71I0635	125		Storm Sewer Headwall							
	City of Eastlake			958477002	Outfall— Jetties	1958	Good	Steel Sheet Pile	9.0	5.5	1/60	Same Structu
Quentin Rd.	10K-9,10	71I0634	125		Pier, Seawall Boathouse		Fair	Concrete Pile Steel Sheet Pile	6		15/40	Same Structu
	10K-8	71I0633	60		Seawall		Fair	Concrete				
	10K-7	71I0632	60		Seawall, Groin		Seawall— Fair; Groin— Poor	Concrete	5.0	3.0	4/30	Same gr
	10M-5	71I0631	100									
	10M-6	71I0630	100									
Woodstock Rd	10M-4	71I0629	175		Shore Return (east) Slabs in Nearshore		Shore Return Fair	Concrete				Concret wall Co groinfi

[1]Property has suffered recession. Bluff face and beach is all that remains. Legal opinion required to determine if privately owned, or now publicly owned.

[2]Bulkhead and seawall are used interchangeably because the exact design of the structures are not known in all cases. Almost all structures are filling the purpose of bulkhead in the strictest sense of the word.

[3]# refers to Carter's report (1976)
XI refers to COE report (1954)

…vation-ft above LWD Shore	Lake	Width/ Length	Previous Structures	Previous Identification[3]	Remarks
					Appears to be no shore return. No tie-in to seawall on 644. Landward fill has pushed top of wall lakeward. Wall is being flanked on east end because no protection on 642.
3.5	3.0	5/30	Concrete Block Seawall Groin-field (4 ea)	#28, XI—70	West shore return ok. No flanking. Old structures incorporated in new structure
					Tied to bulkhead on 641. Extent of east shore return uncertain. 639 property unprotected, flanking likely.
					Groins partially submerged. Extent of shore returns uncertain. No permit or record of previous structures found. Entire structure seems old.
					Source of blocks not known. They make up the beach face for about 100 ft.
9.0	5.5	1/60	Same Structure	#29	Headwall being flanked on west end. No visible accretions of sand on either side of jetties
6		15/40	Same Structure	#30,XI-71	Seawall only behind boathouse—not protection but back to house.
					High up bluff slope. No record of previous structures of permit found. Appears to be old.
5.0	3.0	4/30	Same groin	#31	Elaborate, old terraced seawall very high up bluff. Groin is submerged but still present. Extent of east shore return uncertain. Tied to wall on 633.
			Concrete Seawall Concrete groinfield (3 ea)	#32,XI-72	Shore return is end of next property protection. Slabs may be concrete chunks from old groinfield/seawall. They are arranged in a semicircle shape at east end of property in nearshore zone.

2

C.3. SHORE ZONE

C.3a. GENERAL

The reach is characterized by high bluffs composed of glacial till standing at a vertical angle equal to 50 degrees. At the west end of the reach on 45-J-3,4 (COE, 1954), there is a natural drainage depression of mild slope (Figure 2). From this depression west to the end of the reach, the bluff slope increases gradually. The bluff slope abruptly steepens east of the depression and continues steep to the Eastlake Sewage Treatment Plant. The slope is approximately 25 degrees at the treatment plant because of slope treatment during the construction of the facility.

From the Willowick Shore Club westward to 45-J-3,4, there is an ongoing effort to extend the shoreline lakeward in order to offset recession. This is being accomplished by placing/dumping all types of fill on the bluff face. Photo 1 shows such dumping in progress. The fill, which is not being compacted, is easily eroded. The in-place material is being buried and is no longer visible throughout much of this part of the reach.

In the shore zone, the existing protective structures lie east of the Willowick Shore Club and at the far west end of the reach. Bulkheads are the predominant structure used in this reach. Aerial photography shows that recession of unprotected properties is occurring at a more rapid rate than for protected properties. Table 8 lists the protective structures in this zone.

Photo 1

Dumping Fill on the Bluff

C.3b. IDENTIFICATION OF SOIL PARAMETERS

Field tests of undrained shear strength throughout the reach were attempted with a Torvane meter. In-place material was buried in a significant portion of the reach, so tests could not be performed in those locations. Table 9 shows the observed values and Figure 5 shows the test locations. In order to perform test 4b, 3 inches of till were removed

Table 9. Torvane Readings on 6 November 1978

Location	Height (Ft above IGLD)	Su (Tons/ft^2)
1	14	Material too hard
3	17	0.4
4a	7	0.4
4b	7	0.55(3" back into bluff)
4c	9	0.9
5a	22	0.6
5b	22	0.6(6' left of 5a)
East end	-	Material too hard
West end	-	In-place material covered with fill

from location 4a. The material appeared to be the same in 4b as 4a, the only difference being the amount of moisture present, therefore, it can be concluded that moisture content in the soil significantly affects its undrained shear strength. No conclusion can be reached as to the similarity or dissimilarity of material shear strength throughout the reach based solely on these field tests since the readings vary too widely.

The bluffs were visually inspected to determine if the material was homogeneous throughout the reach. A significant amount of in-place material was not visible, but that which could be seen appeared to be

similar material. There was no evidence of sand lenses or laminated clay. The material color varied from blue-gray to yellow-brown, consistent with weathering of glacial till.

Atterberg limits tests were performed with results shown in Table 10. Samples were taken on 6 November 1978 at locations 1 and 2 on Figure 5. Appendix A contains the test calculations. In order to insure

Table 10. Atterberg Limits Test

Site	NMC(%)	PL(%)	LL(%)	PI(%)	LI(%)= $\frac{NMC-PL}{PI}$	FI	USCS Classification
1	12.3	19.5	28.1	8.6	-0.84	-3.6	CL
2	13.5	19.3	27.5	8.2	-0.71	-4.3	CL

homogeneity, another sample should have been taken at the west end of the reach. However, such a sample could not be taken since the in-place material had been covered with dumped fill. The two samples that were taken can be considered similar at the 5% significance level. Appendix A contains the comparison test for two Atterberg limits tests.

The bluff material is an inorganic clay with low plasticity, medium to high dry strength, medium toughness and none to very slow dilatancy. It has low permeability, therefore it is not susceptible to frost heave. It expands and shrinks with moisture change and is not easily drained. The clay is brittle with the natural moisture content as sampled. An increase in moisture content would increase its plasticity and decrease its cohesive strength.

Considering the above test results and observations, the reach is considered homogeneous throughout its length. There are no obvious material variations that could affect the design parameters for protective structures.

C.3c. GROUNDWATER AND RUNOFF

Runoff over and seepage through the bluff face is controlled differently throughout the reach. At the east end, corrugated pipes are attached to individual storm drains to direct storm runoff to the base of the bluff rather than over the bluff face. Photo 2 shows one application of this method. From the middle to the west end of the reach, the drains protrude out of and discharge directly onto the bluff face. Where fill has been dumped over the bluff face, the drains have been blocked off or covered over, which in turn, causes drainage to discharge directly into the bluff. Photo 3 shows what happens when water drains in this manner. The drainpipe broke or cracked under the force of a slump, and water then drained into the bluff. The soil cohesive strength, therefore, decreased and further slumping was encouraged. The Willowick storm sewer at 45-J-3,4 discharges directly into the lake at beach level.

Photo 2

Drainage Runoff Control

Photo 3

Broken Drain and Subsequent Seepage into Bluff

Because there are very few wells in the vicinity of the reach, the observation well L-1 in Mentor, Ohio (closest observation well) is presented in order to draw some general conclusions. Although the groundwater table elevation is not known without borings being made, Figure 7 indicates that the elevation in the well will generally vary by 2 feet per year, the highest level occurring during the summer and the lowest level during winter. The local population agrees with Figure 7 that the greatest groundwater seepage from the bluff occurs in the spring and summer.

D. METEOROLOGY

The environmental Sciences Services Administration (1979) presents the following description of Lake County climate:

> The climate of Lake County is marked by large annual, daily, and day to day ranges in temperatures. West to northerly winds blowing off Lake Erie tend to lower daily high temperatures in summer and raise temperatures in winter. When winds are from directions other than those mentioned above the presence of the lake has little effect upon temperatures within the rolling to hilly portions of Lake County. Summers are moderately warm and humid in this part of Ohio but temperatures rarely climb higher than 90° F. Winters are reasonably cold and cloudy, but the relative warm waters of Lake Erie temper the air temperatures of on shore winds. Because of this tempering effect, subzero temperatures occur in only 3 of 5 winters. Weather changes occur every few days from the passing of cold or warm fronts and their associated centers of high and low pressures.

LAKE COUNTY

Lake County is in the northeastern part of the State and is bordered on the north by Lake Erie. Two glacial moraines cross the county from northeast to southwest. North of these moraines are several beach ridges of former stages of Lake Erie. Elsewhere, the glacial drift is relatively thin and is composed mainly of till which yields very little water. The underlying rocks are dense shale and limestone which yield only small quantities of water. As a whole, this county is poorly situated for obtaining ground-water supplies.

OBSERVATION WELL L-1

LOCATION - Mentor Water Dept. pumping plant at north limits of Mentor, Mentor Tp., Mentor quad., lat 41°40'42", long 81°19'43", in T. 10 N., R. 9 W., elev. 680 ft. above m.s.l.

DESCRIPTION - Unused drilled well, property of Mentor Water Dept., dia. 8 in., depth 32 ft. Sand aquifer. Measuring point at top of instrument support, 1.85 ft. above land-surface datum. Observation by automatic recorder from April 1948 to the present.

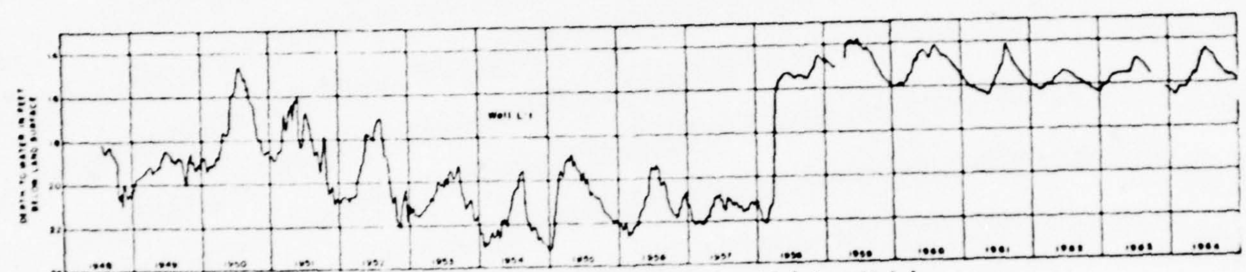
REMARKS - Fluctuations of water level in this well are in response to pumping in nearby wells, which furnish the municipal water supply, and to recharge from local rainfall. During 1951 and 1952, municipal pumping was being increased and as a result the water level declined rather rapidly. Pumping was discontinued or greatly curtailed in December 1957 when Mentor began to obtain water from another source. This was followed by a rise of about 6 feet in the water level in the observation well.

Comparative water-level data for well L-1
(in feet below land surface)

Year	High	Date	Low	Date
1948(a)	18.0	Apr. 14, May 24	20.9	Aug. 5
1949	18.1	Mar. 25	20.2	July 7
1950	14.5	Apr. 11	19.5	Oct. 17
1951	15.5	Mar. 1	21.1	Aug. 31, Sept. 1
1952	16.8	Apr. 23-25	22.2	Aug. 4-6
1953	19.2	June 9,11-13	22.3	Sept. 30
1954	19.2	Apr. 8	23.2	Sept. 30
1955	18.3	Dec. 29	23.3	Oct. 3-8

(a) less than full year of record.

Year	High	Date	Low	Date
1956	19.3	Apr. 16	22.7	Nov. 20
1957	19.7	Mar. 10	22.2	Nov. 11, Dec. 13, 20
1958	14.4	Aug. 24	22.1	Nov. 12
1959	13.6	Mar. 24	16.0	Sept. 30
1960	14.2	Apr. 24-26	16.0	Oct. 5, 6
1961	14.1	May 9	16.4	Feb. 12-16
1962	15.4	Mar. 5,9, 12	16.4	Sept. 26
1963	14.8	Apr. 13,17,19	16.3	Sept. 30
1964	14.4	Apr. 17	16.5	Nov. 9-11



Ground-water levels (lowest weekly) in well L-1.

Figure 7

Observation Well Log, Mentor

As is characteristic of continental climates, precipitation varies widely from year to year, however, it is normally abundant and well distributed throughout the year with winter being the driest season. Painesville's average annual precipitation of 35.68 inches is slightly more than 1 inch below the mean for northeast Ohio. Showers and thundershowers account for most of the rainfall during the growing season. Thunderstorms occur on about 35 days each year. Most of these occur April through August. Over the level terrain of Lake County, most precipitation during the winter months comes from rain but this is not the case 4 to 7 miles southeast of the lake as this area is a part of Ohio's "snow belt". Average annual snowfall within Lake County varies from about 55 inches along the Lake Erie shoreline to more than 90 inches along the Geauga-Lake County line. As is typical of all Ohio, seasonal snowfall in Lake County is subject to wide variations from the annual means.

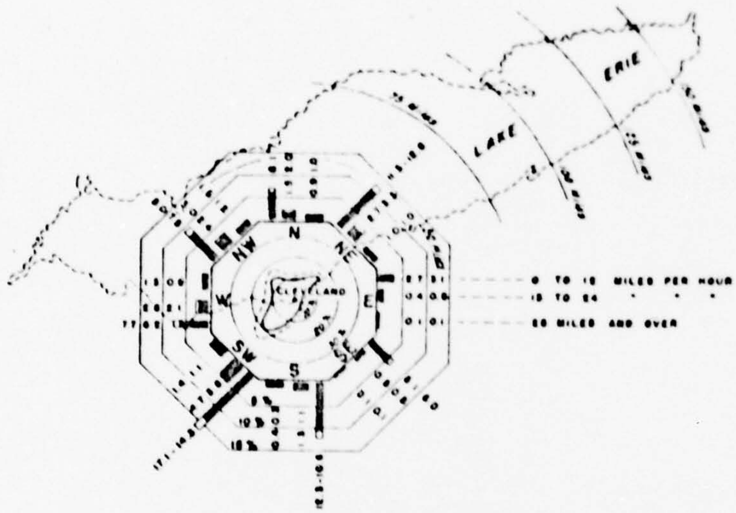
Data gathered at the Cleveland Weather Station is applicable to the reach and Table 11 is a compilation of that data.

Figure 8 and Table 12 show the prevailing wind direction is along the reach from the southwest while Tables 11 and 12 show the strongest winds coming from the western quadrants. There is a secondary maximum wind speed from the northeast. Most wind action, therefore, occurs parallel to the shoreline rather than onshore or offshore.

Table 11. Cleveland Weather Station Data[1]

	Annual	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Max. Temperature (°F)	103	73	69	83	88	92	101	103	102	101	90	82	69
Min. Temperature (°F)	-19	-19	-15	-5	10	25	31	41	41	32	22	3	-11
Mean Number of Days Temperature ≤ 32° F	124	28	25	21	10	1	-	0	0	0	3	12	24
Normal High Temperature (°F)	58.5	33.4	35.0	44.1	58.0	68.4	78.2	81.6	80.4	74.2	63.6	48.8	36.4
Normal Low Temperature (°F)	40.8	20.3	20.8	28.1	38.5	48.1	57.5	61.2	59.6	53.5	43.9	34.4	24.1
Daily Mean Temperature (°F)	49.7	26.9	27.9	36.1	48.3	58.3	67.9	71.4	70.0	63.9	53.8	41.6	30.3
Mean Number of Days Precipitation ≥ 0.01 in.	156	16	15	16	14	13	11	10	9	10	11	15	16
Normal Precipitation (in)	34.99	2.56	2.18	3.05	3.49	3.49	3.28	3.45	3.00	2.80	2.57	2.76	2.36
Average Wind Speed (mph)	10.8	12.5	12.4	12.5	11.8	10.3	9.4	8.7	8.3	9.1	10.0	12.1	12.3
Maximum Wind Speed (mph) and Direction	W74	SW68	W65	W74	W65	SW68	SW57	W65	W61	S45	W43	W59	SW49

[1] NOAA (1978)



WIND DIAGRAM FOR CLEVELAND, OHIO

NOTES

INDICATES DURATION FOR ICE FREE PERIOD (MAR. TO DEC. INCL.) IN PERCENT OF TOTAL DURATION.

INDICATES DURATION FOR ICE PERIOD (JAN. TO FEB. INCL.) IN PERCENT OF TOTAL DURATION.

INDICATES PERCENT OF TOTAL WIND MOVEMENT OCCURING DURING ICE FREE PERIOD.

INDICATES PERCENT OF TOTAL WIND MOVEMENT OCCURING DURING COMBINED ICE AND ICE FREE PERIODS.

FIGURES AT ENDS OF BARS INDICATE PERCENT OF TOTAL WIND DURATION FOR ICE FREE PERIOD AND COMBINED ICE FREE AND ICE PERIODS, RESPECTIVELY.

WIND DATA BASED ON RECORDS OF THE U.S. COAST GUARD AT CLEVELAND, O. FOR PERIOD 1 JAN 1936 TO 31 DEC. 1946 INCLUSIVE.

Figure 8

Cleveland Weather Station Wind Rose (COE, 1954)

Table 12. Measurement of Onshore Winds at Fairport Harbor

(1932 Through 1942)[2]

	Direction of Wind (Percent of total yearly average)				
	SW	W	NW	N	NE
Speed					
0-12 mph	18	7	7	3	9
13-24 mph	5	4	5	2	4
>24 mph	1	2	2	<1	1
Duration	24	13	14	5	14
Movement	22	17	20	6	16

[2] COE (1950)

The U. S. Weather Bureau (1959) indicates the following about storms:

> The more destructive storms on the Great Lakes usually come from a southwesterly direction. These LOWS originally form in three areas:
>
> 1) Texas and New Mexico;
> 2) The Central Rocky Mountains and Great Plains;
> 3) The Pacific Southwest.
>
> The movement of storms from all three regions is similar, from the Middle West to the Great Lakes. The season for storms from these regions is generally from October through May.

Storms coming from the southwesterly direction are consistent with the observed wind directions. Storm data — duration, magnitude, and direction — are not currently compiled by NOAA, but the COE (1954) did compile eleven (11) years of storm data and that information is found

in Table 13.

Table 13. Storms over 18-Hour Duration (1936 Through 1947)[1]

Date	Maximum Velocity (mph)	Direction	Duration in Hours at Velocity >29 mph
03-17-1936	41	NE	19
11-15-1936	45	NW	22
12-27-1938	49	SW	20
04-12 1939	47	W	19
12-07-1939	49	NW	19
01-14-1940	49	SW	19
01-15-1940	47	SW	18
11-06-1940	48	NW	21
03-17-1941	44	W	24
03-09-1942	49	SW	23
03-25-1947	55	W	20

[1]COE (1954)

E. ICE

When the air temperature drops below freezing, spray from waves freezes on the bluff face and on shoreline structures. This ice covering serves as some protection against wave action while the lake remains unfrozen and it will continue as long as the temperature remains at or below freezing. Table 14 shows that freezing temperatures

historically occur during the last half of November and continue into April. The Erie, Pennsylvania record is included to show the homogeneity of temperature on the south shore of the lake. There is no temperature difference, therefore there should be no spatial or temporal variations of extent or duration of bluff face ice armoring.

Table 14. Temperature Data[1]

Location	Normal Daily Minimum Temperature (30 Years Data)		Number of Days When Temperature <32°F	
	November	March	November	April
Cleveland, Ohio	34.4°F	28.1°F	12	10
Erie, Pennsylvania	33.7°F	25.4°F	13	13

[1]NOAA (1978)

Lake ice begins forming at the shoreline and in shallow water, then proceeds toward the middle of the lake and deeper water. Similarly, ice melt starts from the shoreline and shallow water and proceeds to the deeper water. Since Lake Erie is the shallowest of the Great Lakes, it reacts rapidly to seasonal temperature changes, and since heat storage in the lake varies with depth, the lake level will have significant effect on when the lake freezes. At high lake levels, the freeze will occur late and at low lake levels, the freeze will occur early. It is not uncommon for the lake to have complete ice cover during a normal winter.

Complete ice cover protects the bluffs since there is no wave action. Figures 9 and 10 show that shorefast ice protection should last from mid-December through March. A further indication of ice cover is the historical closing of the Port of Cleveland from mid-December through March. There are significant depth differences between the eastern and western basins, but the central basin is homogeneous. The nearshore and offshore depths are consistent throughout the reach so there should be no significant spatial and temporal variations in ice conditions.

Wind and storms may break up existing ice cover. Wind over open water causes convection of warm water from depths to the surface where it melts surrounding ice. Storms and wind blow drift ice toward the downwind shore of the lake where it impacts bluffs and structures, possibly causing damage. This drift ice accumulates causing the downwind shore to have shorefast ice longer than the rest of the lake. Since the predominant wind direction for winter months is southwesterly, the eastern end of the lake is subject to most drift ice impact damage. The entire reach, being parallel to the predominant wind direction, should be subject only to impact damage resulting from local wind direction variation.

The COE is currently studying the feasibility of maintaining year round navigation through Lake Erie. Year round navigation would eliminate the certainty of complete ice cover in normal and heavy winters.

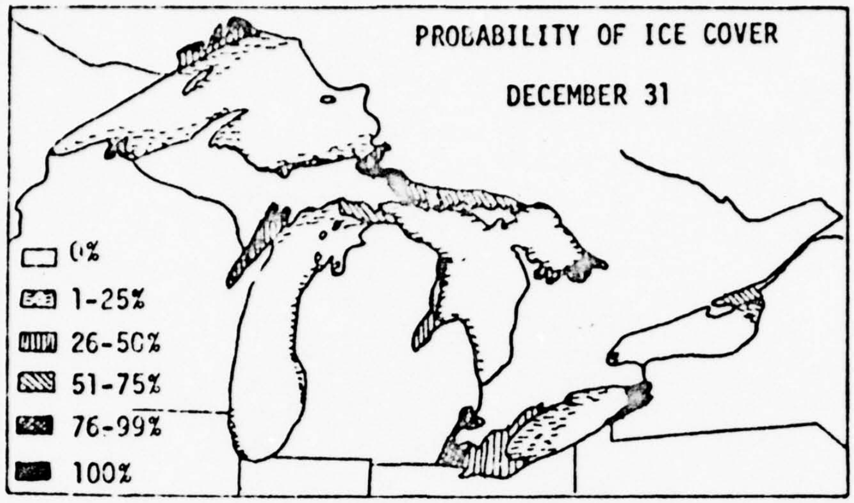


Figure 9

December Ice Cover Map (NOAA, 1974)

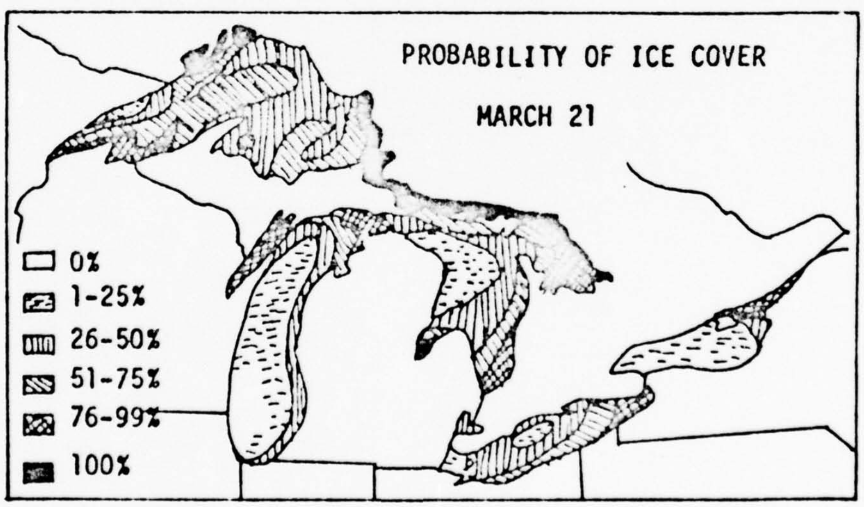


Figure 10

March Ice Cover Map (NOAA, 1974)

But the track made for ships would not be sufficient open water to allow significant wave action. The shorefast ice protection would continue regardless of navigation year round.

The previously mentioned aspects of ice are beneficial towards preventing recession. However, the freezing of water within the bluff does encourage erosion. The bluff material is not susceptible to frost heave, so increase in pressure within the bluff will not be significant, but the freezing of water in small surface cracks and fissures will cause surface erosion. Water in cracks will expand on freezing, thus loosening soil particles or blocks which are then easily eroded by gravity or water action.

F. LONGSHORE TRANSPORT

F.1. QUALITATIVE ANALYSIS

The longshore transport is primarily from southwest to northeast. Evidence for this conclusion is summarized in Table 12 and Figure 8. Table 12 shows that 37% of the yearly average wind comes from a direction (SW, W) that causes transport to the northeast and 19% comes from a direction (N, NE) that causes transport to the southwest. When the wind changes direction as a result of local atmospheric disturbances or meteorological conditions, the longshore transport reverses itself. Inspection of aerial photography confirms that wind direction change affects net longshore transport. 1968 and 1978 photos indicate trans-

port to the northeast while 1974 photos indicate to the southwest. However, the major indicator in the area, the Cleveland Electric Illuminating Company (CEI) water intake jetty, has a sand beach fillet on the western side in all photos, thus indicating a long term net longshore transport to the northeast.

F.2. QUANTITATIVE ANALYSIS

The CEI jetty was constructed at the Eastlake Power Plant in 1952. The 1,080 feet west jetty has accumulated longshore drift since that time. On its west side, material accumulates during eastward transport and is removed during westward transport. Although the jetty is not located in the reach, the change in fillet volume over a period of time is indicative of the minimum net longshore transport rate in the area. The rate may be greater because some material may not be caught by the jetty, some may be lost offshore, and some may be dredged away from the mouth of the inlet structure.

In order to perform the volumetric analysis, 1968, 1974, and 1978 aerial photography was used. A comparative analysis was performed because the original shoreline location was uncertain, the lake level varied between photos, and photography prior to the jetty's construction was not available. The change in volume divided by the number of years between photos is assumed to give the net transport rate. The following simplifying assumptions were made:

a) The jetty height is constant, i.e., no differential settling occurs. This allowed the top of the jetty to be used as a vertical reference point.

b) Achievable accuracy is no better than 1/2 foot. Limitations on the photography, equipment, and operator permit no better accuracy. Since the analysis is comparative, this error should not compound as long as measurements are taken in a consistent manner.

c) The crescent shape of fillet was assumed to be a straight line as seen in Figure 11. The offshore slope is the same as beach face, so a comparative analysis will minimize this error.

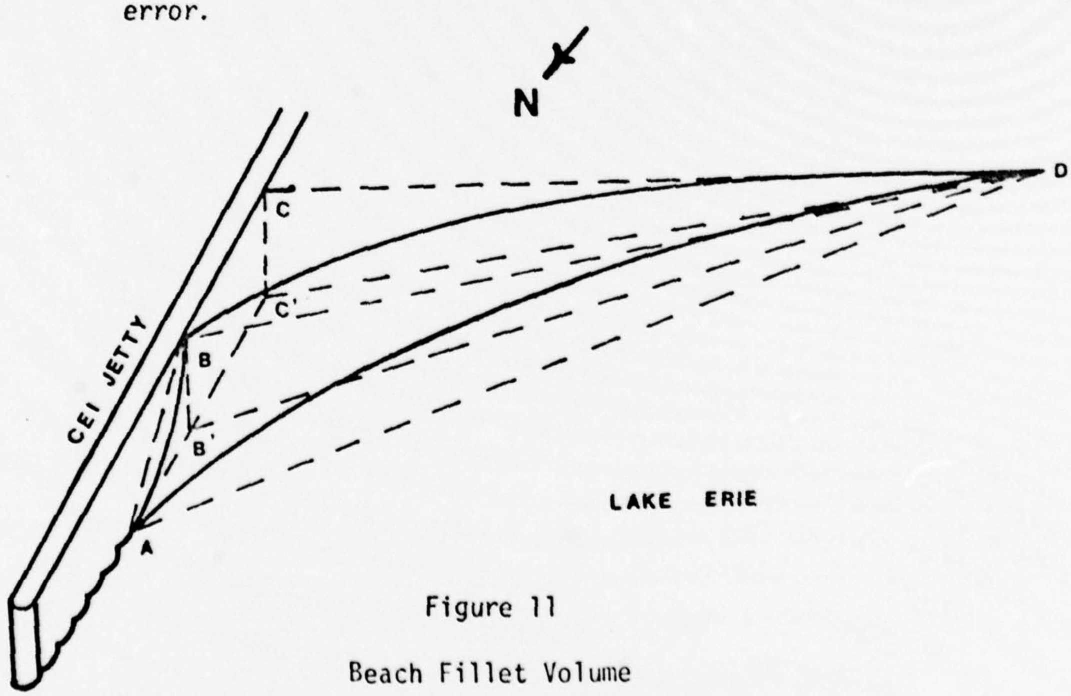


Figure 11

Beach Fillet Volume

The 1968 and 1974 photography was taken from 2400 feet with a Zeiss 152.64 mm camera. The similarity of 1978 photography to 1968 and 1974 allows a conclusion that its altitude was the same. The parallax formula that determines elevations relative to a fixed or assumed point (beach/water interface elevation = 0) is:

$$\Delta h(\text{ft}) = \frac{\Delta p(\text{mm})\ Ha(\text{ft})}{b(\text{mm})}$$

where Δh = change in elevation from the fixed point, Δp = parallax reading from parallax bar, Ha = altitude (2400 ft), and b = length of conjugate axis (distance between principal points on two adjacent stereo photos). Vertical measurements were made with the mirror stereoscope and the parallax bar. Horizontal distances were determined by scaling photos using the known length (1,080 ft) of the west jetty. Appendix B contains the calculations for gross and net longshore transport rates.

The net longshore transport rate is 14,000 yd^3/yr. The gross longshore transport rate lies between 44,000 and 140,000 yd^3/yr, a difference of about 100,000 yd^3/yr. The percentage of wind durations used to calculate gross rate varies with different years of record, different locations, and with inclusion or exclusion of ice periods. Table 12 probably included ice periods which should not be considered in longshore transport analysis. Determination of an input rate would indicate the magnitude of gross longshore transport rate.

The recession rate, as determined by Carter's study of aerial photography, gives an indication of the input rate into the longshore transport. The volume removed from the bluff is carried away by the longshore transport, and therefore the gross longshore transport should be at least that amount. Table 15 shows the recession rates and input rate for the reach.

Table 15. Input to Longshore Transport

Range	Recession Rate (ft/yr)		Length (ft)	Average Bluff Height (ft)	Volumetric Recession Rate (ft^3/yr)	
	Min.	Max.			Minimum	Maximum
2-3	0	1	2,300	41	0	94,300
	3	5	1,900	41	233,700	389,500
3-4	3	5	600	41	73,800	123,000
	1	3	1,000	41	41,000	123,000
	0	1	2,400	41	0	98,400

Total input rate = 348,500 (min) to 828,200 (max)

38,000 yd^3/yr < Gross longshore transport rate < 92,000 yd^3/yr

The calculated gross longshore transport rate of 44,000 to 140,000 yd^3/yr will readily include the input rates of 38,000 and 92,000 yd^3/yr. However, more bluff material may be fed into the longshore transport than is available from this reach, therefore input may be much higher than indicated and well towards the 140,000 yd^3/yr figure. For this reason and in order to be conservative, a gross longshore transport

rate on the order of 100,000 to 140,000 yd^3/yr is estimated.

The gross longshore transport rate is more critical for this design consideration than the net transport rate. The net rate consists of removal and accretion, whereas the gross rate is removal only. The net transport rate indicates how a beach can be maintained by the return of sand previously lost. However, unlike a sand beach, material lost from a bluff cannot be placed back into the bluff. Although the entire gross transport of 140,000 yd^3/yr may not be removed from this particular reach, the fact that material is removed at all is critical to bluff recession.

The longshore transport rate should be applicable throughout the reach because there are no major structures or headlands to disrupt the transport. A better analysis could be performed with more photography and with more sophisticated photogrammetric techniques. In addition, on site monitoring could provide additional data. A thorough search through various agencies might produce transport rates for neighboring reaches which could be extrapolated to this reach.

CHAPTER FOUR

SHORE EROSION PROCESSES

SHORE EROSION PROCESSES

A. RECESSION RATE

The recession rates in Carter's report (1976) are based on a comprehensive analysis of maps and aerial photography. The change in shoreline location divided by the number of years between photos allows a determination of recession rate. The results of that analysis are found in Table 16. Stanley (1969) presents a qualitative analysis of the severity of erosion based on a site inspection. The analysis correlates well with Carter's recession rates.

Table 16. Recession Rate Analysis[1]

Location	1876 - 1937		1937 - 1973	
	Recession Rate[2]	Length[3] (ft)	Recession Rate[2]	Length[3] (ft)
Ranges 2-3	Slow	3,950	Very Slow	2,300
			Moderate	1,900
Ranges 3-4	Slow	550	Moderate	600
	Moderate	650	Slow	1,000
	Slow	2,750	Very Slow	2,400

[1]Carter (1976)

[2]Very Slow, < 1 ft/yr; Slow 1-3 ft/yr; Moderate, 3-5 ft/yr

[3]Measured to nearest 50 ft.

B. WAVE EROSION

One major cause of recession in this reach is wave action. Surface waves generated by wind blowing across the lake expend much of their energy in eroding the nearshore, beach and shoreline, and shore zones.

The nearshore slope in this reach is less than one degree, and because the wave breaks when the water depth is about one to two times the wave height, the waves have lost a great deal of energy by the time they reach the shoreline. Because the beaches are narrow, little energy is absorbed by them. In light of the above, storm waves do the greatest damage. Because of their large height and high frequency, storm waves retain greater amounts of energy to be expended on the shoreline than swell waves.

Due to the continuous removal of material from the base of the bluffs, the slope cannot reach long-term equilibrium. Recession continues as long as waves attack the bluff toe.

C. SLOPE STABILITY

C.1. GENERAL

Another major cause of shoreline recession is slope instability. Slope instability, known as mass-wasting in geologic terms, is the erosion of soil blocks rather than soil particles. An increase in moisture content decreases the soil strength. When the strength becomes too little to hold the slope, failure occurs and the soil block

slides downslope. Wave action then erodes the material that has accumulated at the bluff toe.

C.2. TYPES OF FAILURE

Slope stability analyses are performed in order to determine the type of failure that occurs throughout the reach and the likelihood of failure at any location. Knowing the predominant failure mechanism, a protective scheme can be developed to prevent that failure.

A soil's plasticity is indicated by its LI. Figure 12 shows the stress-strain relationship for soils with different LI's. If the

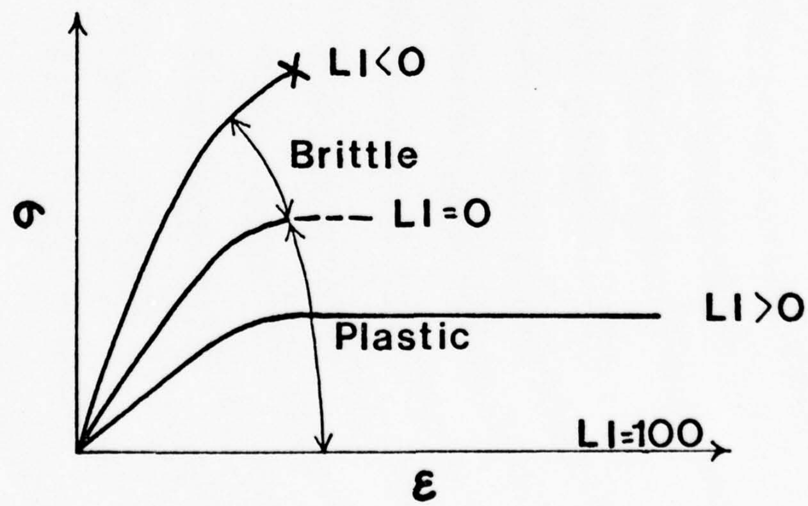


Figure 12

Soil Plasticity as f(LI)

NMC is below the PL, the soil is in its brittle state. When strained, it loses its strength when it fractures at its peak value. If the NMC lies between PL and LL, the soil is in its plastic state. When strained, it increases its strength to a lower peak value and then maintains that strength. If the NMC is above LL, the soil has little strength because it is in its liquid state. Figure 14 is a graphic illustration of failure types related to NMC.

Brittle failure occurs if NMC is below PL. To illustrate the failure type, consider a vertical slope with differential elements A-E lying on the probable failure surface as shown in Figure 13. Point A will be stressed greatest because of its location. If it is stressed until fracture, point B will not fracture at the same time because it has not been stressed as much as point A. When point A fails, point B will have to take up the stress until it fractures, and so forth until

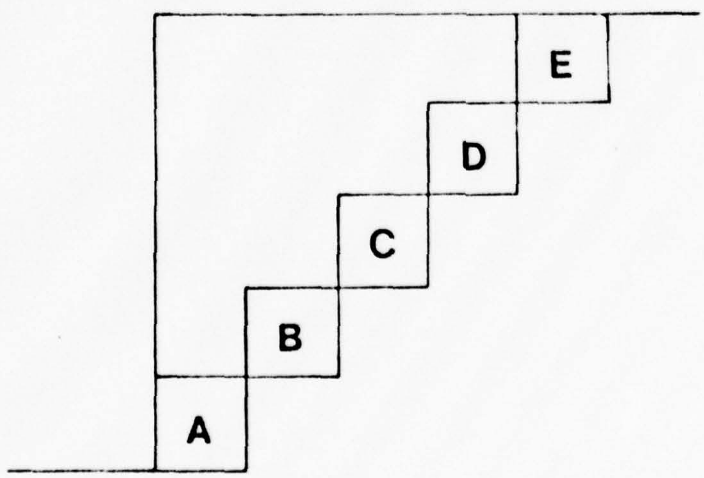


Figure 13

Slope Failure

all differential elements have failed. A brittle failure is also known as a progressive failure. Due to its progressive nature, block falls are characteristic of brittle failure. When point A fails, everything above it is likely to fall eventually.

Plastic failure occurs when NMC lies between PL and LL. The LL cannot be attained because the soil strength would be insufficient to sustain a stable slope. The value that can be attained will be dependent on the soil properties. To illustrate the failure type, consider Figure 13 again. Point A will be stressed to its peak value and will hold that value rather than fail. Point B will increase to its peak value after point A has reached its peak. This continues until all points are sharing the stress at the peak value they can sustain. If any further stress is added, all differential elements will fail simultaneously and a sliding (plastic) failure will occur. Plastic failures are characterized by sliding or slumping, and may occur slowly or rapidly.

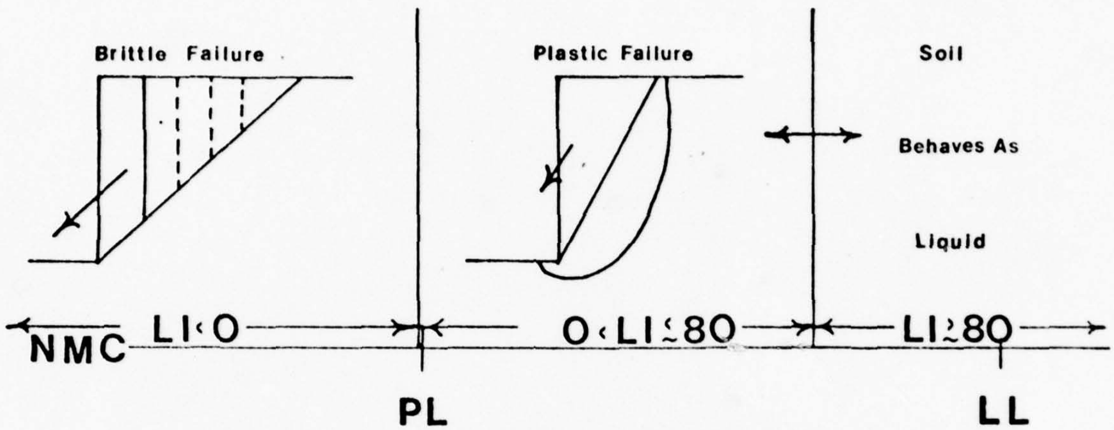


Figure 14

Failure Type as f(NMC)

C.3. APPROACHES TO ANALYSIS

Once the failure type is determined for the soil, the existing slope must be analyzed to determine its likelihood of failure. The factor of safety concept is used and is defined as:

$$FS = \frac{\Sigma \text{ Forces resisting failure}}{\Sigma \text{ Forces causing failure}} \qquad \text{C.3.-1}$$

The vertical and finite slope methods are used for this analysis.

The vertical slope method assumes a homogeneous, purely cohesive soil. Using a FS = 1 and a plane failure surface, an analytical solution for a stable bluff height can be found. If a brittle condition exists, the bluff can stand at $H_B = \frac{2c}{\gamma}$ and if a plastic condition

exists, the bluff can stand at $H_B = \frac{4c}{\gamma}$.

The finite slope method (method of slices) allows stratification in the soil and differences in moisture content (presence and location of groundwater). The analysis is performed on a curved arc failure surface to determine the FS of the existing bluff. For a plane surface failure, an analytical solution can be found for the height of the bluff as in the vertical slope method.

In order to perform these analyses, the soil weight and cohesion must be known. For those analyses that result in a specified height, the actual height is compared to the calculated height and a determination is made as to whether the slope is approaching failure or is stable. If H actual < H_B, the slope is stable. For the method of slices, the FS is calculated and the determination is made whether the slope is stable or unstable.

While developing analytical solutions in the above methods, an interesting and important fact surfaces. The weight of the soil above the failure surface not only causes the failure, it also resists failure. A component of the weight goes to each action, but the component causing failure is usually greater than the component that resists failure. This is due to the frictional nature of soils.

C.4. ANALYSIS OF EXISTING SLOPES

Atterberg limits and NMC were the only two tests performed

on the bluff material. For a more thorough analysis, as for final design work, further tests would be required to determine soil parameters, changing moisture contents, groundwater table variations, and other appropriate data. For the preliminary design, conclusions will be drawn from the completed tests, observations and literature.

The bluff material is a low plasticity clay with a LI << 0. The material is very brittle for the existing November 1978 moisture content. For the condition as tested, brittle failure of the slope should be expected, i.e., block falls and progressive failure.

However, visual inspection of the bluffs does not show any evidence of block falls. On the contrary, recent and old slumps are common throughout the reach as shown in photos 4 and 5. Stanley (1969) indicates no slope stability failures at all and Carter (1976) indicates slumps occur in the reach, but not block falls.

Although the LI << 0 thus indicating brittle failure, photos 6 and 7 show that the till can and does reach plastic conditions. In photo 6, the till has obviously flowed in a plastic condition. Photo 6 shows the till in its plastic condition. The NMC in the bluff at these locations was high enough to increase the LI of the fill to its plastic state.

The calculated NMC is representative only of November 1978. According to Table 11, spring and summer are the months with most precipitation, therefore soil moisture content would be the greatest at

Photo 4

Slump Failure

Photo 5

Slump Failure Caused by Overburden

Photo 6

Plastic Flow of Till

Photo 7

Till in a Plastic Condition

that time. This is further verified by local residents who indicate the bluffs are very "wet" in the spring. Determination of a year's variation in NMC would allow a better conclusion concerning the predominant soil condition.

With soil parameters unknown, an analysis cannot be completed on height or FS. However, a general conclusion can be made. For brittle soils, $H_B = \frac{2c}{\gamma}$ and for plastic soils, $H_B = \frac{4c}{\gamma}$. Cohesion decreases with increasing moisture content. The preponderance of evidence indicates that plastic failure is most common. The clay may lose so much cohesion when plastic that it will offset the increase in γ and the factor of 2 in the coefficient. Appendix C shows the change in γ from PL to LL. The actual bluff height may in fact be too high in the plastic condition but sufficient in the brittle condition to preclude failure. The presence of slumps and no block falls seem to indicate that this may be true. With the evidence available, it appears that the bluffs readily change from the brittle to plastic condition. Furthermore, slump failure, not progressive failure, appears to be the critical design condition. Moisture control is therefore extremely important!

The above conclusion is applicable to in-place material. The unconsolidated fill being placed on the west end of the reach will erode swiftly by surface runoff and wave action. Once that material is removed, the bluffs should then act in accordance with the foregoing conclusions.

D. SLOPE EROSION

Although rainfall, ice, and drainage do cause erosion, they do not play as significant a part as do wave action and slope stability. Impact of rainfall and its associated runoff erode soil particles. Alternate freezing and thawing of ice in small cracks (expansion and contraction) disrupts the surficial deposits. Surface drainage, natural or from storm drains, enters cracks and causes pressure build-up which can loosen soil particles making them more susceptible to erosion. The soil particles eroded by these methods are carried directly into the lake by runoff or deposited at the toe of the bluff.

CHAPTER FIVE

LIMNOLOGY

LIMNOLOGY

A. LAKE LEVEL

Lake level elevation is normally given with respect to International Great Lakes Datum (IGLD). IGLD was defined in 1955 as mean water level in the Gulf of St. Lawrence at Father Point, Quebec. Care must be taken when comparing lake level elevation and surface elevation. USGS topographic maps use reference datum of mean sea level (MSL) measured at Sandy Hook, New Jersey. In order to make elevations consistent, add 1.6 ft to IGLD or subtract 1.6 ft from MSL. For example: the lake elevation of 568.6 ft (IGLD)will result in inundation of shoreline to 570.2 ft (MSL). Lake Erie datum is 568.6 ft (IGLD) or 570.2 ft (MSL).

The point of application of wave action on the bluff face is dependent on the lake level. When the lake is at high stage, erosion is at its maximum. To design for the worst condition expected in the life of the structure, a lake level and wave height must be determined. The North Central Division, COE, designs shore protection for a 200-year event. The 200-year event is used in this design in order to agree with COE methods.

Historical data on lake levels have been compiled by the Detroit and Buffalo Districts, COE. Figure 15 is the Buffalo data based on period of record 1904-1970. Figure 16 is the Detroit results based

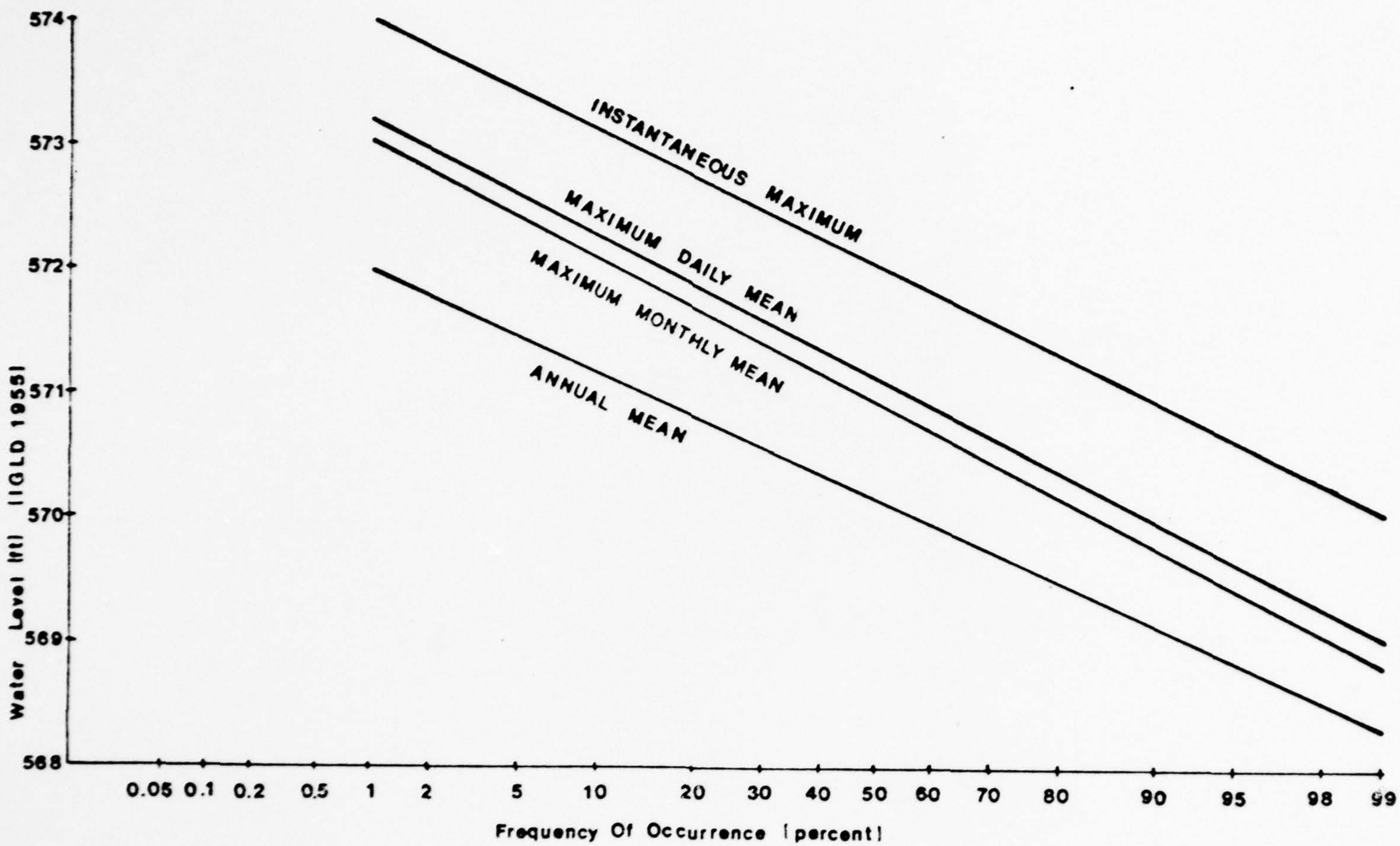


Figure 15

Lake Erie Annual Stage Frequency Curves at Gordon Park, Cleveland (COE, 1978)

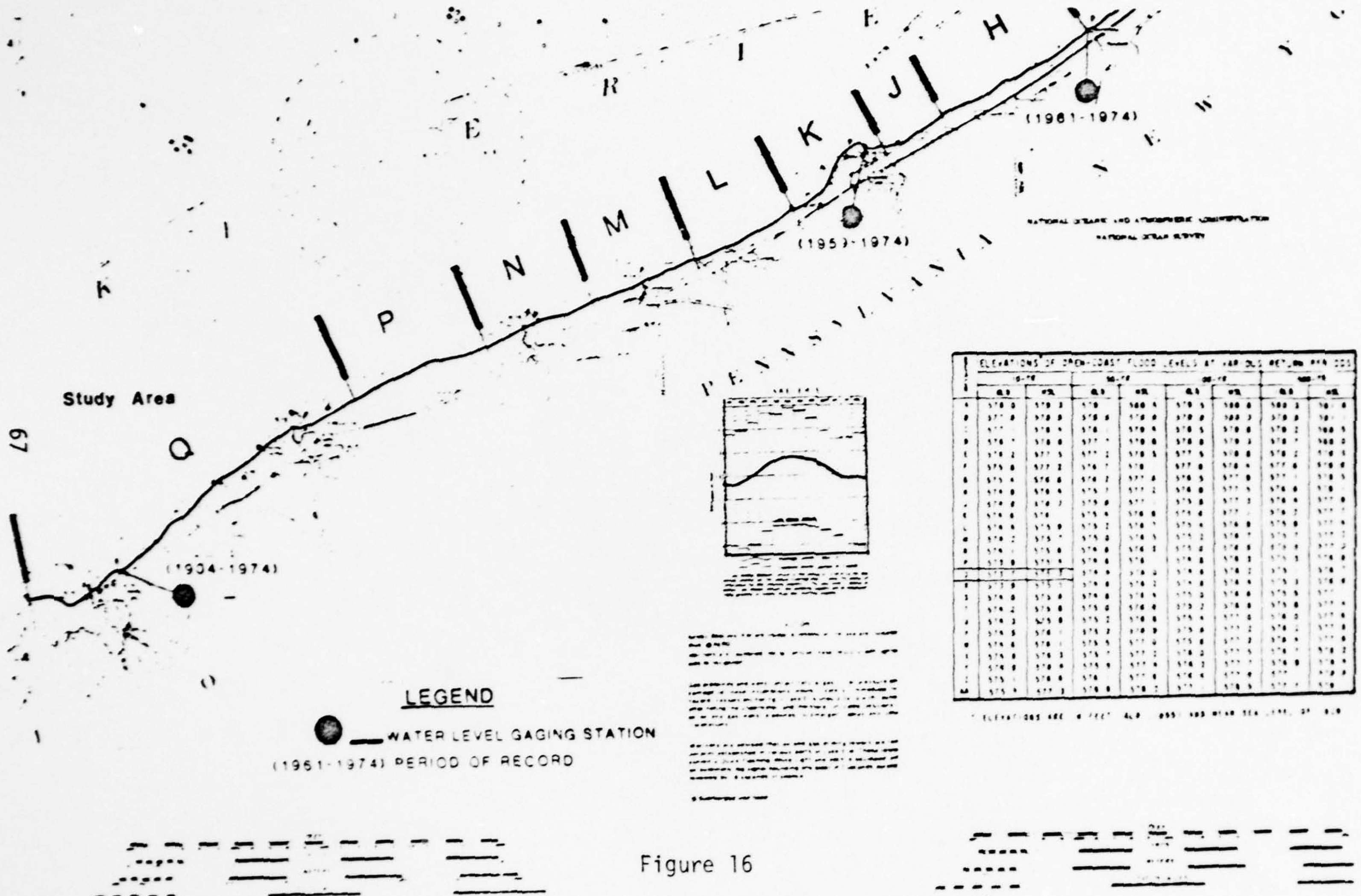


Figure 16

Maximum Annual Lake Levels, Detroit (COE, 1977)

on period of record 1899-1974. For the same return period, Buffalo's level is approximately 1/2 foot lower than Detroit's level. This is accounted for by the length of period of record and inclusion of high lake levels from 1970 to 1974. Detroit data is used due to larger data sample and a higher (conservative) lake level.

There are two acceptable methods on determining lake levels. One method is to add short term fluctuations (seiche, storm surge, set up) to the maximum monthly mean for the recurrence interval. The second method is to use the instantaneous maximum lake level for the recurrence interval. The second method, Figure 16, is the method currently accepted by the COE. The second method is used in this report for the following reasons:

a) Short term stage variation would be interpolated from recorded data that is not available for this reach.
b) Stanley (1969) shows that the reach is at or near the primary nodal point of longitudinal oscillations in the lake.
c) Other modes of oscillation cause level changes, but since they are not as significant as longitudinal oscillations, time and effort is not warranted in a preliminary design analysis for developing those changes.
d) Historical short term fluctuations are included in the instantaneous maximum lake levels.

Using Figure 16, the lake level for a 10-year return period is 573.7 ft (IGLD) or 575.3 ft (MSL).

B. WAVE ANALYSIS

Wave action actively erodes the shoreline. The height of waves determine the extent of erosion. The prevailing wind and depth conditions in the reach determine the wave heights. The worst conditions produces a wave height that is the design wave.

Resio and Vincent (1976) contains deepwater wave heights and periods for various locations and return periods. The 20-year return period is used in order to arrive at a 200-year event (20-year design wave + 10-year instantaneous maximum lake level). Table 17 lists the parameters for the reach. Winter conditions are not considered since ice armoring occurs during that period and wave action is minimized. Refraction analysis must be accomplished in order to correct the deep water wave to the inshore condition.

Refraction analysis is based on bathymetric chart #14825, NOAA. The depth contours were corrected for the 10-year lake level. Since refraction occurs at 1/2 Lo, the shallow lake will cause refraction throughout its area. Refraction diagrams are completed for the three worst conditions which occur in the fall. Figures 17, 18, and 19 show the refraction diagrams and Appendix D contains the calculations. Comparison of refracted wave height and breaker height indicates broken waves are the design condition. The design breaker height is therefore used as the design wave height. This design wave height does not include runup since that is a function of the structure. The design wave height is 2 1/2 ft (rounded up from 2.3 ft). Figure 20 is a cross section view of the design elevations.

Table 17

Wave Parameters at Shoreline Gridpoint #11, East of Cleveland[1]

		Angle of Approach[2]		
		1	2	3
Spring	H (ft)	5.6	7.5	8.9
	T (sec)	5.98	6.5	7.26
	Lo^3 (ft)	184	216	273
Summer	H (ft)	7.2	7.2	8.5
	T (sec)	6.46	6.38	7.1
	Lo^3 (ft)	216	210	258
Fall	H (ft)	10.5	11.8	10.8
	T (sec)	7.4	7.84	8.02
	Lo^3 (ft)	280	312	328

[1]Resio and Vincent (1976) - 20-year return period.

[2]Angle 1 - angle > 30° to right of normal to shore;
Angle 2 - angle within 30° to either side of normal to shore;
Angle 3 - angle > 30° to left of normal to shore.

[3]Table C-3, Shore Protection Manual (SPM, 1975)

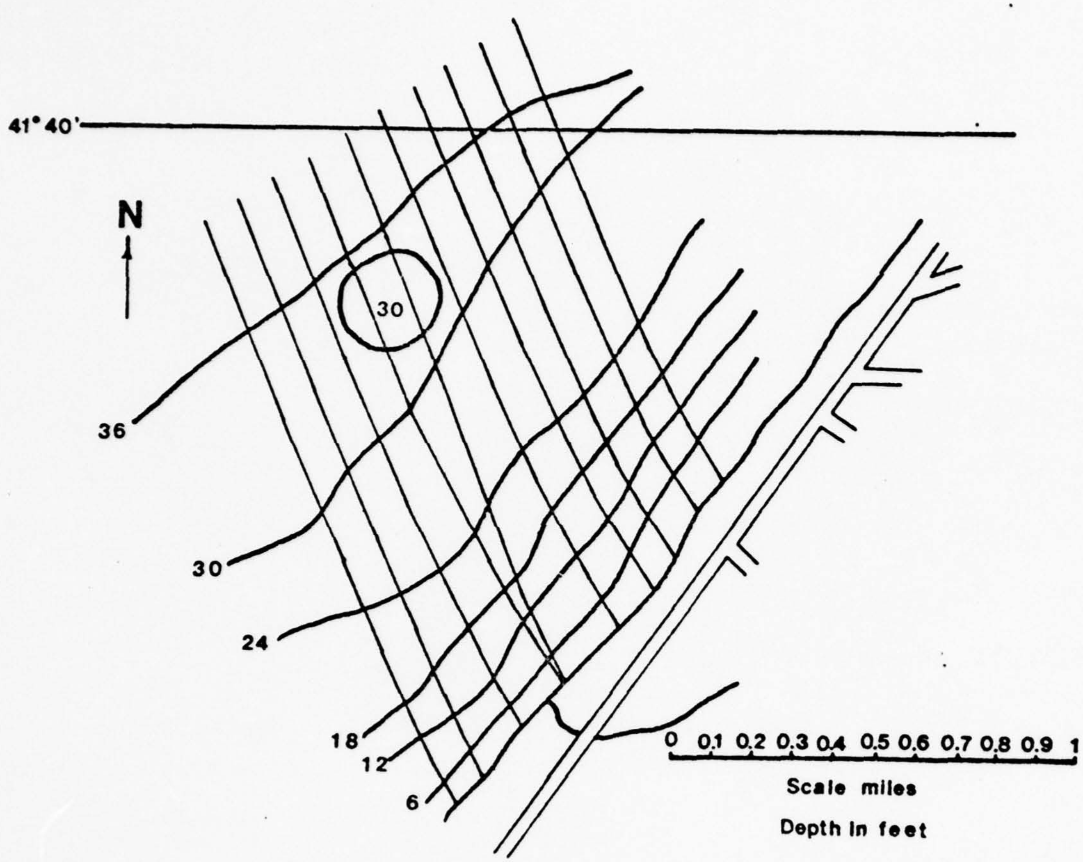


Figure 17

Angle 1 Refraction Diagram

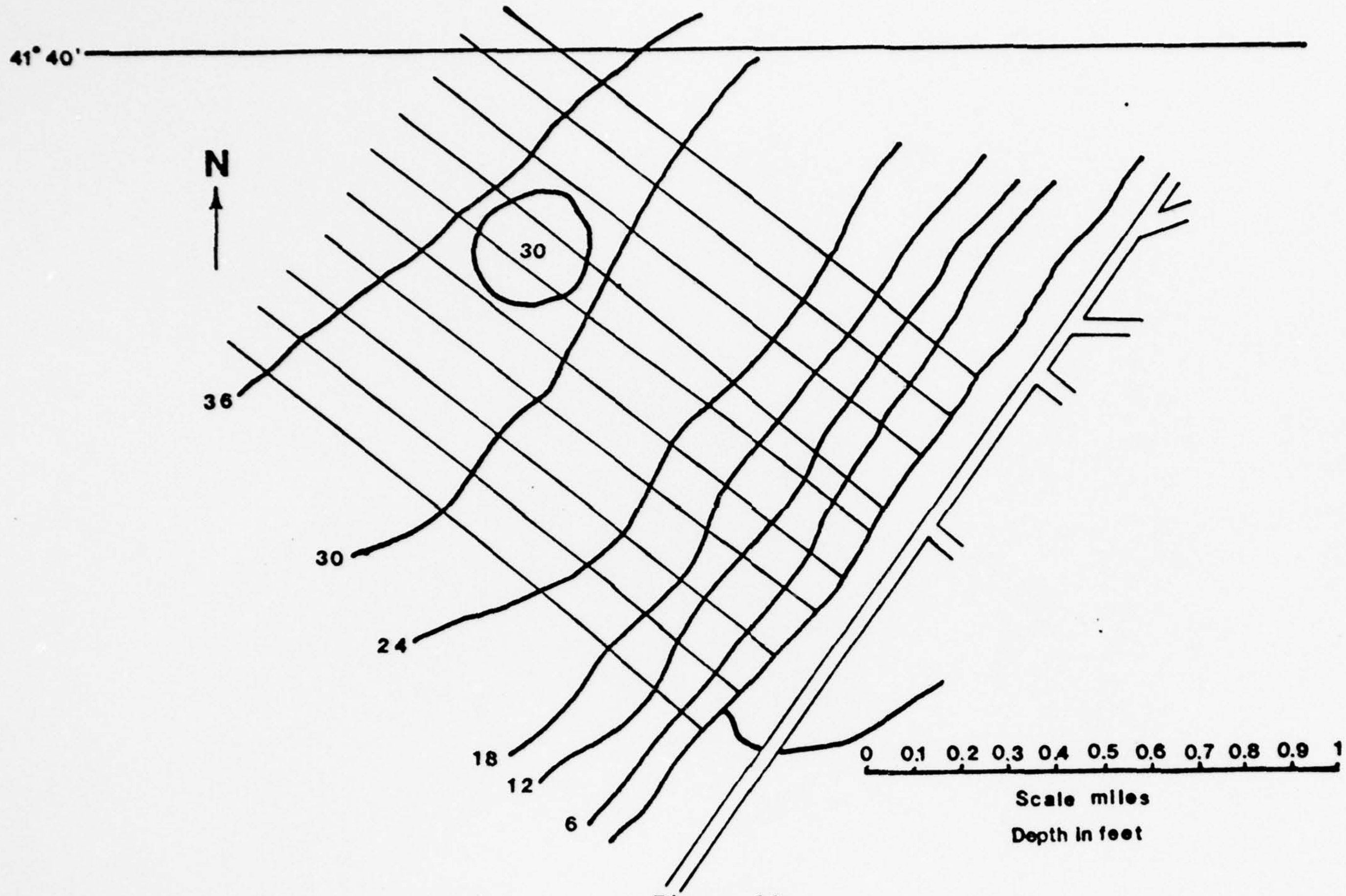


Figure 18

Angle 2 Refraction Diagram

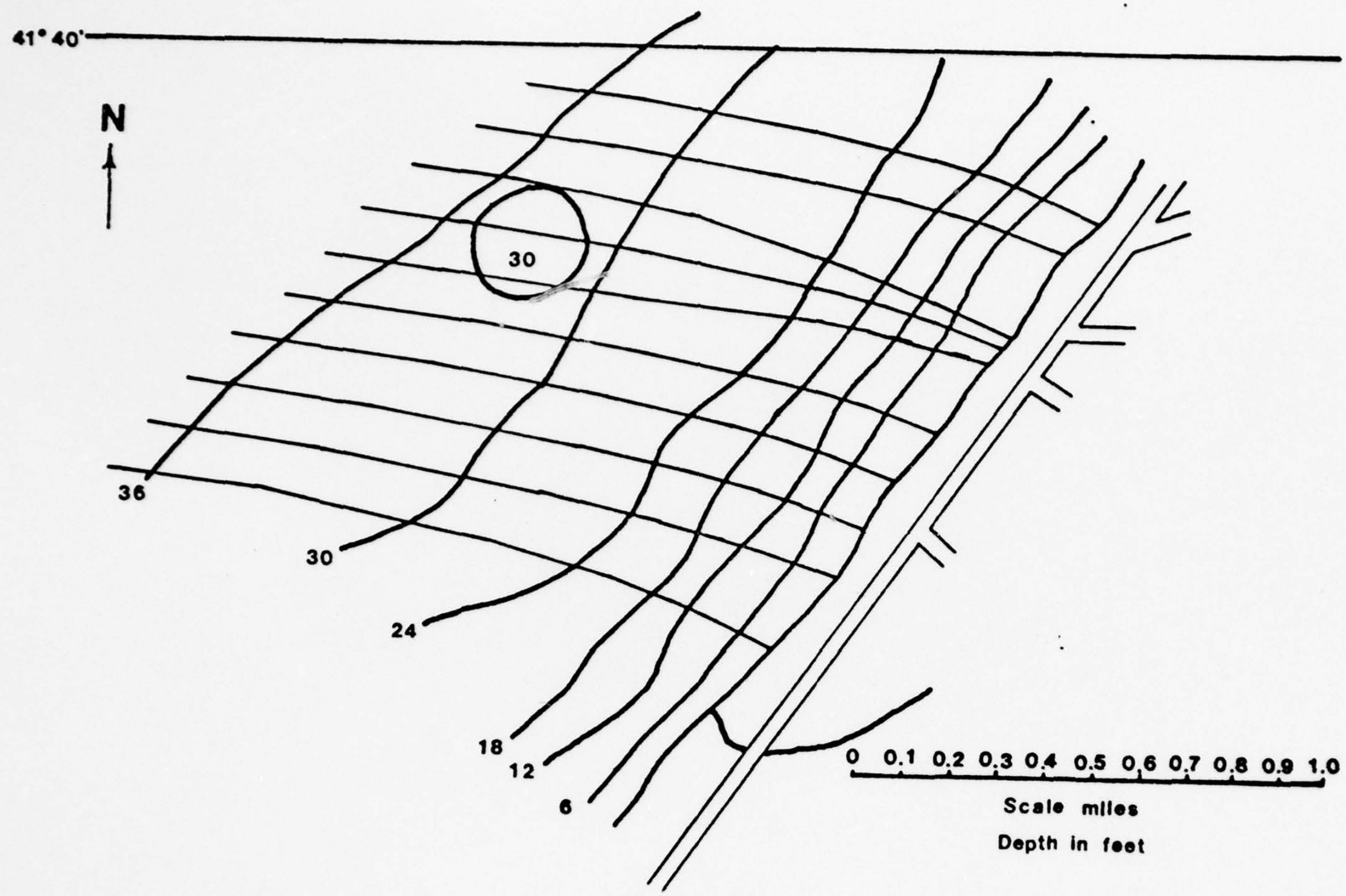


Figure 19

Angle 3 Refraction Diagram

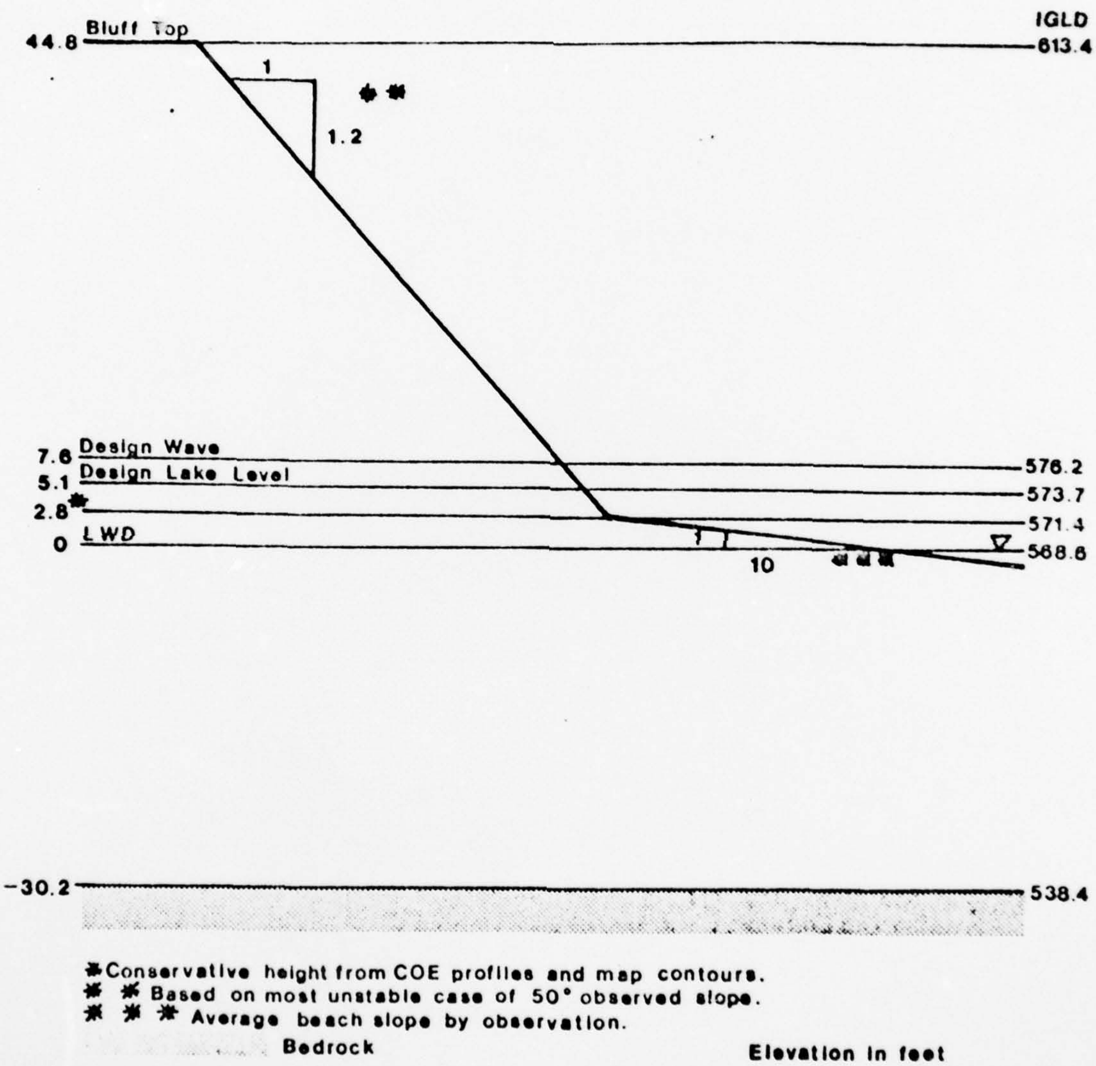


Figure 20

Design Elevations

CHAPTER SIX

DESIGN CONSIDERATIONS

DESIGN CONSIDERATIONS

A. GENERAL

The combined effects of bluff instability and wave action result in bluff recession in this reach. In order for any protective system to be successful, it must stabilize the bluff as well as control wave erosion of the toe. Recession will continue if both are not included in the design.

Toe protection alone may fix the bluff toe location. However, the bluff top will continue to recede until a stable slope angle is reached. This would probably be acceptable were it not for the proximity of structures to the bluff. Significant damage would occur before long term stability could be reached. Stanley (1969) suggests toe protection with and without stabilization. The design with stabilization is very expensive and the design without stabilization does not halt recession.

Slope stabilization alone would also be unsuccessful. Although the slope could be stabilized at some angle, wave action would continue to remove material at the toe. This would constantly weaken the base of the slope, eventually resulting in slumps or slides and recession would continue unabated.

B. DESIGN CRITERIA

The design should be as low cost as possible. Materials that are readily available in the area should be used. Hand placement of materials by private owners, CETA workers, municipal employees, etc., should be employed whenever and wherever possible, rather than machine placement. Existing structures should not be removed but should be integrated into the design in order to avoid the costs of building new structures.

Lake County, the cities of Eastlake and Willowick, or the landowners themselves should be able to effect the design. It should be simple enough that cooperation, shared cost, and joint effort, at whatever political level, will insure successful installation. A complex design might discourage the landowners from organizing to implement it should the cities or county be unable to administer it with their engineering resources and expertise.

It is assumed that beach recreation is not high priority in this reach. This conclusion results from several observations:

a) There are no public thoroughfares to the beach.

b) Fill, sometimes completely covering the beach, is being placed on the bluff and beach by private property owners as well as the City of Willowick.

If this assumption is incorrect, design modifications can be made to allow for beach accretion. These modifications may or may not be signi-

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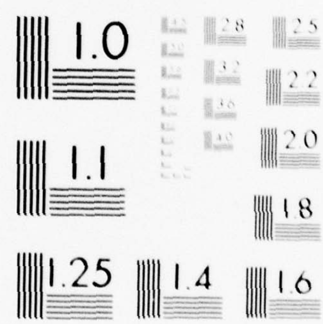

MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

ficant depending on the amount of beach that is required, but the cost with such modifications will probably be higher. Some landowners have existing structures that trap sand or encourage beach building and these beaches should remain since existing structures will be integrated into the final design.

Interruption of littoral transport or littoral supply is not a major consideration in the design. Since the CEI power plant jetty completely cuts off the littoral transport, the downdrift reach east of the design reach receives no littoral input now. Changing or interrupting the littoral transport within the reach, therefore, should not increase erosion in the downdrift reach.

C. NON-FEASIBLE ALTERNATIVES

Contrary to the conclusion of Brater et al (1977), beach accretion-type structures, groins in particular, would not significantly limit recession. Brater's laboratory results show that recession is limited but the demonstration groins, although accumulating some beach, did not eliminate recession and slumping of the bluff due to slope adjustment. Anderson et al (1978) noted that the bluff along Chesapeake Bay receded a great distance even when the beach had widened considerably. According to Anderson, a low longshore transport rate (100,000 to 140,000 yd^3/yr is in that range) is insufficient to establish large beaches, so large scale, expensive nourishment would be

required just to maintain the beach. The accumulated beach would not stabilize the bluff but would serve only as toe protection. COE (1954) suggested building a beach as toe protection and allowing the bluff to stabilize. Houses were far enough from the shoreline at that time to allow the bluff to reach its natural slope, but they are too close today.

Wave-damping structures are represented by offshore breakwaters, sills (perched beach), floating breakwaters (used tires or otherwise), and artificial seaweed. Again, the longshore drift is insufficient to build a beach in the shadow zone of these structures and massive breakwaters are too expensive. Floating breakwaters, though inexpensive, are effective only for small waves with short periods as pointed out by Ross (1975). Artificial seaweed does not effectively attenuate wave energy at periods commonly found in large bodies of water (COE, 1976). As with groins, these structures do not serve to stabilize the slope but only protect the toe.

Vegetation, although a complementary part to any design, cannot prevent recession in the reach. Although vegetation decreases slow types of mass movement such as soil creep, it cannot slow the deep slumps and slides which prevail in this area. The bluff is too steep for vegetation growth. However, if the slope is flattened somewhat, vegetation might grow and therefore minimize surface erosion. Toe protection would still be required.

The current practice of dumping landfill from the top of the bluff is not a feasible long term means of halting erosion. If soil is used, the fines are rapidly washed away and coarse material continues to erode at the toe. If rubble, concrete or rock, is used, it armors somewhat, but the haphazard placement and lack of filter at the toe allows the fines to continue to wash away and toe erosion to continue. Landfill of trash, junk cars, stoves, and similar objects are objectionable for aesthetic as well as pollution reasons. In addition, they allow toe erosion to continue for the same reasons as concrete and rock. Landfill, if drained, compacted, protected with filter, placed properly and in conjunction with a toe protection structure, can successfully halt recession. But haphazard dumping simply feeds fines to the lake and destroys the aesthetic appeal of the shoreline.

Slope excavation for slope stability is not feasible for several reasons. Cutting back to a stable slope would involve tremendous earthmoving expenses. The houses are too close to the edge to allow cutting back without relocation, an unaccceptable alternative. The bluff is too high and steep to allow benches to be cut. Stanley (1969) suggests adding fill lakeward 25 to 50 ft, protecting the toe, and then allowing the bluff to stabilize. This method is too expensive at the 1969 price of $217 per linear foot.

D. FEASIBLE ALTERNATIVES

Although relocation of buildings from the shoreline is a feasible solution, it is assumed that it would not be an inexpensive nor popular solution. Therefore, group effort with structural measures will be considered as the most acceptable solution.

Bulkheads/seawalls and revetments are the most likely structures to halt recession in this reach. Not only do they protect the toe against wave action, they also help to stabilize the bluff. A bulkhead, as defined in the SPM (1975), prevents sliding of the land as well as protects the upland from wave action. A seawall, on the other hand, protects the upland from wave action as well as prevents sliding of the land. Revetments serve principally to protect against wave action, though the more massive they are, the more they stabilize the slope. COE (1954) suggests heavy riprap revetment to protect against recession. All three structures, though defined differently, basically fulfill the same functions.

In order for these alternatives to be successful, several precautions must be taken. In the case of seawalls and bulkheads, adequate tie backs must be insured to prevent toppling due to back pressure. In all cases, adequate shore returns must be guaranteed to prevent flanking and toe protection must be provided to prevent scour.

To insure stability of the slope, proper drainage must be provided. A drainage design is necessary to complement the structural

alternatives at the toe. Stanley (1969) does not address drainage requirements as part of slope stabilization. Carter (1976) does suggest adequate drainage in addition to toe protection in the form of a wall, while COE (1954) suggests a tile drainage system at the top of the bluff.

CHAPTER SEVEN

DESIGNS

DESIGNS

A. GENERAL

Preliminary designs of toe protection and drainage are presented in this chapter. A detailed engineering design would require further tests and study. A combination of toe protection and drainage should protect the toe against wave action and stabilize the bluff. These alternatives emphasize low cost and design simplicity.

A major factor in maintaining low cost is the use of waste material as construction material. Significant quantities of rock, broken concrete, masonry, bricks, and soil are discarded along the bluffs as shown in Photo 8. The material currently in place is free and need only be picked up and used for whatever purpose necessary. Similar waste materials from nearby construction sites may be available at little or no cost. The use of these waste materials in the protective structures eliminates the requirement for purchasing new construction materials.

Gabions used in these structures are compartmentalized boxes of steel wire mesh filled with rock of sufficient size to be retained by the wire mesh. They are relatively inexpensive and can be hand emplaced. Since gabions function like building blocks, they are easily adapted to any design configuration and when placed and filled properly, the resulting structure is sturdy. The permeability of a gabion is at once

Photo 8

Gabion Fill Material

a drawback as well as an advantage. The gabion disperses wave energy and does not reflect wave energy directly towards the toe as does an impermeable seawall. Therefore, scour at the toe is minimized. But the permeability does allow fines to wash through the structure. The leaching of fines from the bluff decreases its stability so filter

fabric must be used to line the bottom and back of a gabion structure. Galvanized steel and polyvinyl chloride coating minimizes the possibility of failure due to corrosion and abrasion. Should failure occur, however, only one compartment will be affected and it can be repaired by refilling the compartment and patching with a new sheet of mesh. If not promptly repaired, further damage will occur due to differential settling, so the gabions should be inspected after storms and on a recurring basis.

The suggested construction materials are standard size and prices quoted are 1978 prices unless noted otherwise. Bekaert Gabions of Reno, Nevada, provided the information on gabions. The 1978 fully installed estimate is based on a 1977 quotation for northern Ohio which was corrected by an average inflation rate of ten percent per year. The filter paper used is Poly-Filter X produced by Carthage Mills of Cincinnati, Ohio. Medusa Aggregates, Columbus, Ohio, provided the cost estimates for gravel/rock and Building Materials and Specialties, Columbus, Ohio, provided the cost estimates for drainage pipe. Final cost estimates may vary widely from those stated dependent upon supplier, implementation time, etc.

B. GABION SEAWALL

Figure 21 is a cross section of the first alternative, a stepped, gravity seawall constructed of gabions. The structure height, including runup of 3.2 ft, is 576.9 ft (IGLD). Runup calculations, found in

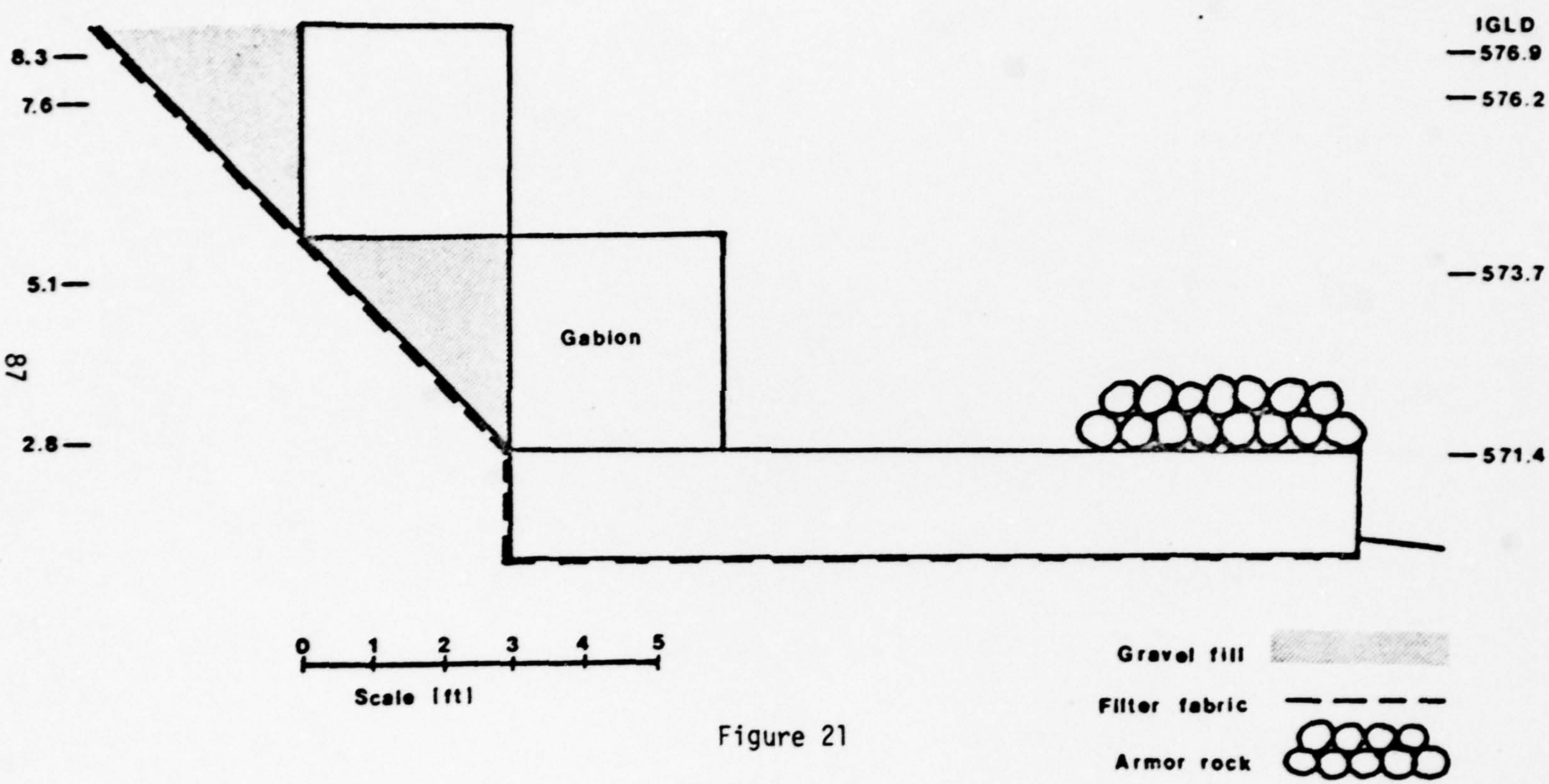

Figure 21

Gabion Seawall

Appendix E, are based on a riprapped slope since gabions filled with rock act like a riprapped surface.

Due to mesh size, the gabions are filled with 4 to 10 inch stone or the waste materials, concrete, bricks, and masonry. A horizontal shelf is cut before placing the gabion mattress at the toe, thus insuring a stable foundation for the wall. The longitudinal axes of the wall and mattress gabions are perpendicular, thus further enhancing structural stability. The gabion placement is staggered for stability, as when laying bricks, and all gabions are laced together. The effectiveness of lacing is improved if accomplished while gabions are empty.

Filter fabric is placed under and behind the structure because it prevents the leaching of fines from the bluff face and the scouring of sand from the structure toe. Gravel behind the wall allows free drainage of groundwater, surface runoff, and overtopping spray. Fifteen (15) inch armor rock, as calculated in Appendix E, is placed at the toe of the gabion mattress to protect against damage by floating debris.

Since the standard gabion length is 12 ft, Table 18 shows the material required per 12 ft of beach front. The resultant cost for materials per foot of beach front is $37. This figure assumes there is sufficient waste material to provide all the gabion fill. The fully installed estimate (including aggregate and labor) is $116 per foot. The availability of free or cheap materials and unskilled labor (CETA

Table 18. Gabion Seawall Materials

Gabions				Filter Fabric		Gravel[1]		Toe Rock[2]		Gabion Fill
12x3x1 1/2		12x3x3								
Number	Unit Cost	Number	Unit Cost	Amount	Unit Cost	Amount	Unit Cost	Amount	Unit Cost	
4	\$47.30	2	\$68.15	360 ft^2	\$.105/ft^2	5.7 ton	\$2.90/ton	13.5 ton	\$4.50/ton	16 yd^3

[1] $\left[\frac{120\ lb/ft^3}{2000\ lb/ton}\right]^{-1} = 0.06\ ft^3/ton$; Unit cost does not include hauling.

[2] $\left[\frac{155\ lb/ft^3}{2000\ lb/ton}\right]^{-1} = 0.08\ ft^3/ton$; Unit cost does not include hauling.

workers, owner construction, etc.) will determine where the actual cost will lie between $37 and $116.

This alternative involves manual labor except for cutting the mattress shelf and hauling materials to the site. The use of a bucket loader would, however, speed up and simplify the filling of the gabions.

C. GABION REVETMENT

The second alternative, a step-sloped gabion revetment, is shown in Figure 22. The structure height, including a 2.7 ft runup, is 576.4 ft (IGLD). Since gabions act like a riprapped surface, runup calculations in Appendix E are based on a riprapped slope.

A horizontal shelf is cut for the gabion mattress, thus insuring a stable foundation for the structure. The longitudinal axis of mattress and structure gabions are perpendicular to further stabilize the revetment. The gabions are laced together, filled with 4 to 10 inch waste material that can be retained by the mesh, and staggered, as in the laying of bricks.

Filter fabric is placed under and behind the structure to prevent scour and leaching. A 6 inch gravel blanket placed on the filter fabric allows free drainage from the bluff face while another layer of filter fabric then prevents the compacted fill from leaching away.

The fill, selected from the waste material, should be relatively coarse and permeable and should be compacted to prevent differential

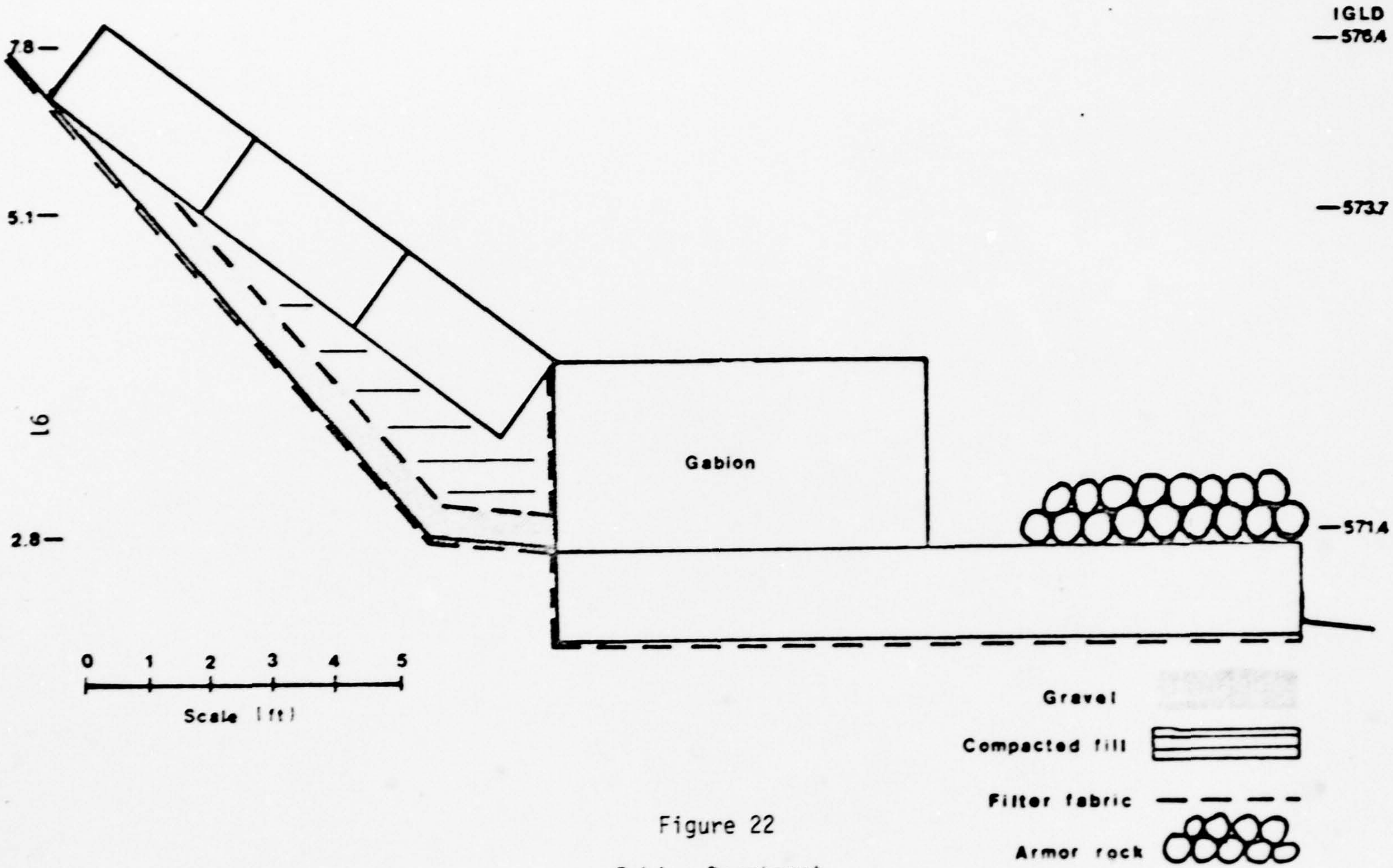

Figure 22

Gabion Revetment

settling of the gabions placed on the slope. Non-slaking shales and limestone may be used if available. The fill need not be soil, but may be the broken concrete, masonry and bricks used as gabion fill, provided that the gabions are already filled and excess material remains. Clays should be avoided for the reasons already mentioned in Chapter Four.

Table 19 shows the material required per 12 ft of beach front, the standard gabion length. The resultant cost per foot of beach is $51. This figure assumes there is enough waste material to fill the gabions and to supply all the required fill. The fully installed estimate (including aggregate and labor) is $158 per foot. Depending upon the availability of cheap or free materials and labor, the cost of this structure will lie between $51 and $158.

Some machine work is necessary for this alternative. A mattress shelf must be cut, material must be hauled, and fill must be compacted. The movement and placing of materials would be simplified by the use of a bucket loader.

D. ROCK REVETMENT

The final alternative, a sloped rock revetment, is shown in Figure 23. The revetment is a standard type, except for the core of waste materials. Use of this waste material saves the price of an equivalent amount of quarry rock, but there may be insufficient waste material to

Table 19. Gabion Revetment Materials

Gabions				Filter Fabric		Gravel[1]		Toe Rock[2]		Gabion Fill	Fill
12x3x1 1/2		12x3x3									
Number	Unit Cost	Number	Unit Cost	Amount	Unit Cost	Amount	Unit Cost	Amount	Unit Cost		
7	\$47.30	4	\$37.85	552 ft^2	\$.105/ft^2	4.5 ton	\$2.90/ton	13.5 ton	\$4.50/ton	22 yd^3	18 yd^3

[1] $\left[\frac{120\ lb/ft^3}{2000\ lb\ ton}\right] = 0.06\ ft^3/ton$; Unit cost does not include hauling.

[2] $\left[\frac{155\ lb/ft^3}{2000\ lb/ton}\right] = 0.08\ ft^3/ton$; Unit cost does not include hauling.

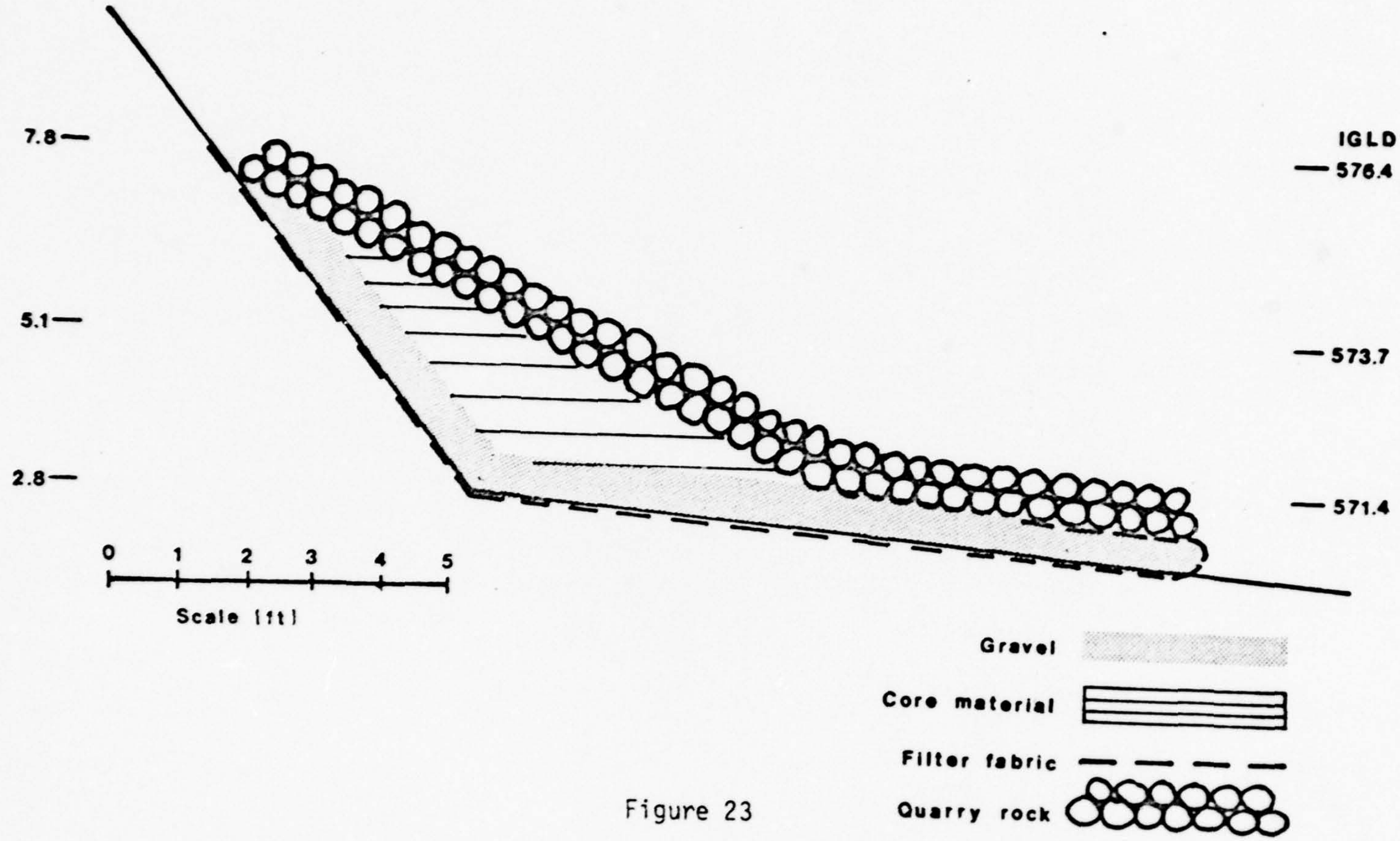

Figure 23

Rock Revetment

fill the core, so some quarry rock may have to be purchased. If this material is placed so that unit size increases from bottom to top, the revetment should be as stable as if quarry rock entirely were used in the core.

Filter fabric is placed directly onto the natural profile followed by a 6 inch gravel blanket, thus preventing the leaching of fines from the bluff. The gravel blanket also provides a base upon which to place the heavier core material. To hold the gravel in place, the filter fabric is turned back and anchored by the armor rock at the toe.

The revetment, having a stable 1 on 2 slope, causes a 2.7 ft runup resulting in a 576.4 ft (IGLD) design height. A 3 ft layer of 150 to 300 lb armor rock is required according to Hudson's formula. The toe is extended 4 ft to protect against scour. The armor rock and runup calculations are found in Appendix E.

Table 20 shows the materials required per 12 ft of beach and their unit costs. The 12 ft length allows direct comparison with Tables 18 and 19. The cost per foot of beach is $21 if sufficient waste material is available to fill the core. If only quarry rock were used in the core the cost would be $27, so the total cost of materials for this alternative is $21 to $27. There will be a significant increase in these costs once the machine and skilled labor costs are added. This alternative requires a crane and trucks for hauling, but little manual labor.

Table 20. Rock Revetment Materials

Filter Fabric		Gravel[1]		Rock[2]		Core Fill
Amount	Unit Cost	Amount	Unit Cost	Amount	Unit Cost	
240 ft^2	\$.105/ft^2	7 ton	\$2.90/ton	45.1 ton	\$4.50/ton	200ft^2

[1] $\left[\dfrac{120\ \text{lb/ft}^3}{2000\ \text{lb/ton}}\right]$ = 0.06 ft^3/ton; Unit cost does not include hauling.

[2] $\left[\dfrac{155\ \text{lb/ft}^3}{2000\ \text{lb/ton}}\right]$ = 0.08 ft^3/ton; Unit cost does not include hauling.

E. DRAINAGE

A proper drainage system removes water from within the bluff and diverts surface runoff so that it does not erode the bluff face. The suggested toe structures allow free drainage from the bluff face, thus precluding the buildup of hydrostatic pressure. A subsurface drainage system at the toe should improve the flow of porewater, but there is insufficient information concerning costs, equipment, and groundwater to develop such a design. A reasonable alternative, based on available information only, is a subsurface drainage system at the top of the bluff interconnected to the existing surface runoff system. This design could be refined if borings and groundwater indormation were available.

Figure 24 shows a cross section of the proposed drainage system. A gravel filled drain is used because it draws surface water as well

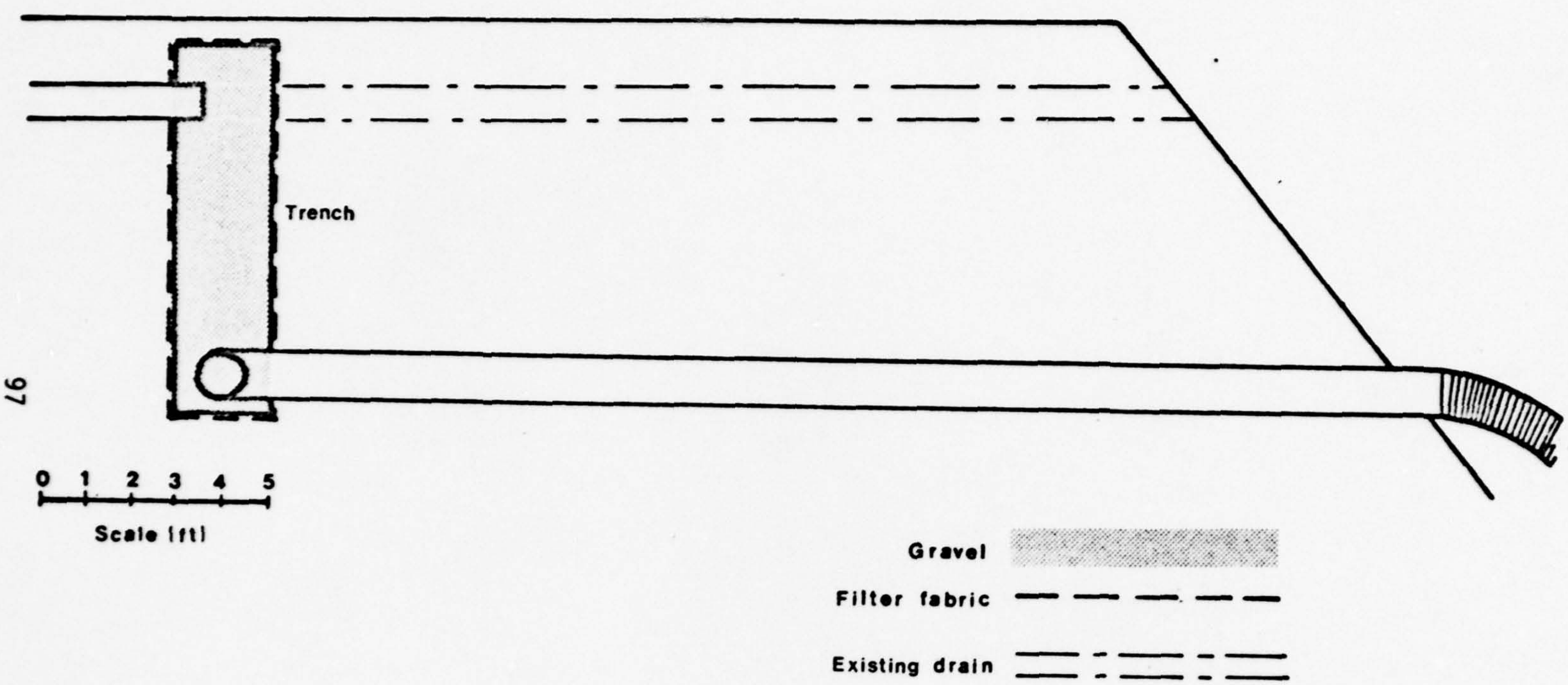

Figure 24

Drainage System

as porewater. The drain is surrounded by filter fabric to preclude the necessity for a graded gravel filter. The drain must be 2 ft wide in order to accomodate the 10 inch perforated flexible pipe. The existing drains may be connected directly to the collector pipe if at the correct depth. When the existing drains are at an incorrect depth, they should be cut and the part lakeward removed and placed at the new depth as shown in Figure 24. Since lightweight equipment (backhoe) must be used so close to the bluff edge, the depth of excavation is limited by the equipment capability. An 8 ft depth is used as an average value, but a deeper drain is prefereble if the equipment available has the capability. Flexible pipe, as shown in Photo 2, is used from the storm drain outlet to the bluff toe in order to divert runoff from the bluff face. The drain is placed 20 ft back from the bluff edge in order to allow for safe construction, to remain effective should recession continue, and to remove surface water before it percolates into the bluff. Photo 9 shows typical surface water that must be removed.

Table 21 shows the costs of materials per foot of beach (assuming storm drains every 50 ft) with a resultant cost per foot of beach of $11. These costs do not include labor or machine costs, so the actual cost will be somewhat higher. Since these costs assume all new pipe, any storm drain that can be directly interconnected or reused will decrease this cost accordingly.

Photo 9

Standing Water on Top of Bluff

Table 21. Drainage Materials

Filter Fabric		Gravel[1]		10 Inch Flexible Pipe (Perforated and Unperforated)	
Amount	Unit Cost	Amount	Unit Cost	Amount	Unit Cost
18 ft^2	\$.105/ft^2	1 ton	\$2.90/ton	2 ft	\$3/ft

[1] $\left[\frac{120\ lb/ft^3}{2000\ lb/ton}\right] = 0.06\ ft^3/ton$; Unit cost does not include hauling.

Drains must be properly maintained in order to continue functioning correctly. If a pipe breaks, as has occurred in the past, it must be promptly repaired. If not, it will leak water into the bluff, thus decreasing stability and increasing bluff recession.

F. COMPARISON OF DESIGNS

F.1. COST

A complete design alternative consists of toe protection and drainage system. Table 22 shows a comparison of maximum and minimum costs of each alternative. Installation costs (labor and equipment)

Table 22. Design Cost Comparison

Design	Drainage and Structure Cost ($ per ft)	
	Minimum	Maximum
Gabion Seawall	48	127
Gabion Revetment	62	158
Rock Revetment	32	38[1]

[1]Does not include installation.

are not included in the minimum costs but, except for the rock revetment, are included in the maximum costs. When installation of the rock revetment is considered, its maximum cost will increase significantly since it is the most equipment intensive alternative of the three.

The quoted costs are strictly estimates and actual costs may vary widely dependent upon numerous factors (supplier, transportation, time, etc.).

F.2. STRUCTURAL STABILITY

Lacing gabions together in the seawall is difficult due to limited overlapping of contiguous baskets. The gabion revetment has more abutting surfaces, therefore lacing is improved. Additionally, since back pressure tends to cause toppling and/or sliding of walls, the gabion revetment better resists sliding since the gabions are placed with the strong long axis countering that force. The rock revetment has a stable 1 on 2 slope and gains stability through the interlocking of individual stones whereas the gabion structures each have a steeper composite slope.

F.3. BLUFF STABILITY FACTORS

All three alternatives have gravel/filter blankets that allow free drainage from the bluff face. The revetments add significant weight lakeward at the toe of the bluff which serves to stabilize the bluff by increasing the resistance to slumping action.

F.4. SCOUR

All three alternatives have protection against scour at the toe. The gabion mattresses and the rock revetment extension are all flexible

and extend some distance lakeward from the structure toe. As scour occurs under them, the lakeward end will settle but the portion under the structure will remain stable and unaffected. Furthermore, once the sand scours away to glacial till, further scour will be limited because till is highly resistant.

F.5. DAMAGE RISK

Gabion structures in the Lake Erie environment have not proven totally satisfactory to date (personal communication with Denton Clark, 21 March 1979, and Fred Ball, 20 March 1979). The structure foundation cannot be rubble or sharp objects, but must be smooth and stable or else the structure will rapidly fail. It is difficult to ensure the careful attention of unskilled laborers to properly filling the baskets and lacing them together. Structures subject to storm waves are very susceptible to damage from debris carried by wave action and significant damage may result from a single storm, thus requiring almost complete rebuilding of the structure. The gabion seawall, therefore, is less subject to damage than the gabion revetment due to its increased distance from the wave action.

The rock revetment is not very susceptible to damage because the armor rock protects against debris carried by storm waves. Armor rock must be placed carefully to ensure interlocking, but such care is more likely from the skilled equipment operator than from the unskilled laborer handloading and lacing gabions.

CHAPTER EIGHT

APPLICATION

APPLICATION

The configurations of the suggested alternatives were developed based on an average cross section of the reach. But the profiles and conditions vary throughout the reach, therefore the design must be slightly modified to account for these variations. Construction limitations and integration of existing structures must also be considered when applying an alternative. All three toe protection alternatives are similar in that they are seawall/revetment type structures and therefore will be similar in application. A general application follows with references to special problems of any alternative. Figure 25 shows a plan view of the design application.

The drainage ditch must be placed the length of the reach, except at location "a" at the far west end. The slope is very mild and vegetated at that location, so current drainage is sufficient. Additional drainage may be required at location "b" if very much fill is placed in the ravine. All drainage should be interconnected to existing storm drains whenever possible.

The heights of all existing structures must be considered. Those structures higher than design height require no special consideration, but those lower than the design height must be built up or merely incorporated at their existing height. Whenever possible, the existing structures should be built up to design height. The simplest method

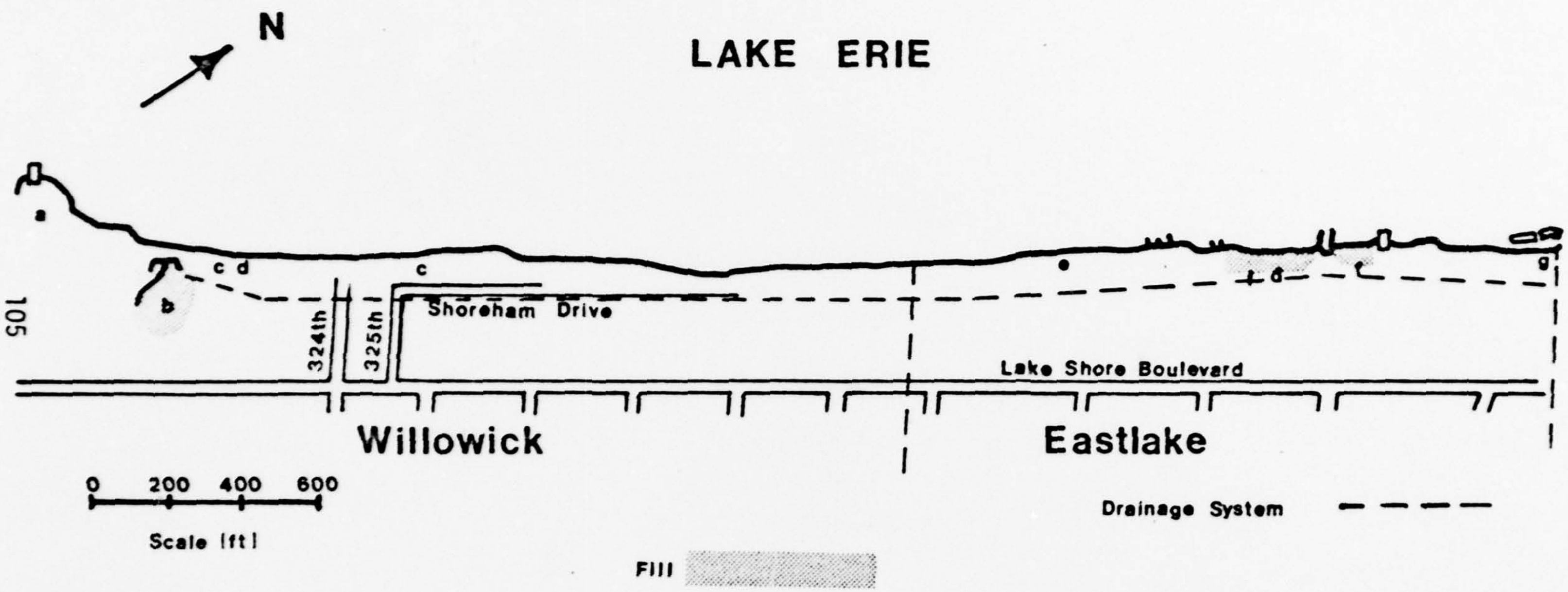

Figure 25

Design Application

of increasing height is to armor the area above and behind the existing structure to design height. This can be done with armor rock or with gabions, depending on the alternative selected. If the old structure is upgraded with the same material used in the new structure, total construction effort will be simplified.

At locations "c" toward the west end, the toe of the bluff has been extended lakeward by dumping fill. If the new structure is constructed as the toe now exists, it would jut out from the neighboring portions and would also be in the water. The toe structure should not be built directly in the water and ought to follow a smooth alignment. Therefore, some of the fill material must be removed.

Fill is required at locations "f" in order to bring the receded profile into alignment with the predominant profile. This is necessary where the shoreline has been unprotected and a concave shape has resulted. Additionally, fill is required at location "b" to fill a natural depression. The bluff profile must be at least as high as the design height in order to provide support at the rear of the structure.

There are concrete slabs/slope revetments at locations "d" that can be incorporated into the new structure. The in place concrete can be used as the foundation and/or toe protection for the new structure. Any loose, heavy rock in the water can also be rearranged to give added protection to the new structure.

The Willowick Shore Club seawall of concrete-filled drums at

location "e" must be rebuilt or replaced. The drums must be rearranged and placed upright on a foundation with toe protection. Some fill material behind the wall may have to be removed in order to properly reconstruct the wall. The drums should be reused, since they can be effective if placed properly.

The new structure should be tied to existing structures. If gabions are used, lacing to old structures will stabilize the new structure. Small holes must be drilled in old structures to receive the lacing. The rock revetment structure must be abutted carefully against old structures. Regardless of selected alternative, the scour protection device should continue to the front of the old structures. This continuous scour protection will tie all of the structures together.

A shore return is required at location "g". This shore return should be substantial, since it is the only one required in the reach and it will anchor the east end. A cut of 10 to 15 ft should be made into the bluff for the placement of a gabion wall. Since rock revetment does not lend itself to a shore return, some type of return wall must be constructed for it. A gabion return wall would be satisfactory since it is cheap, easy to install, and could be easily tied in to the rock revetment.

CHAPTER NINE

CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS AND RECOMMENDATIONS

Shoreline recession is a landward retreat of the shoreline. The retreat occurs at the top of a bluff as well as at the toe. Wave action causes the retreat at the toe. But slope instability, in addition to wave action, causes the retreat at the top. Unfortunately, most coastal engineering literature concerns inself with wave action and fails to consider slope stability. Both physical mechanisms must be considered in any study of bluff recession. If a bluff is stabilized and its toe protected, recession can be effectively limited.

Carter's report (1976) was undertaken in order to provide an informative reference to assist in making decisions related to shoreline erosion abatement. It is, with minor exceptions, an adequate document for preliminary design work. The bedrock contour map was greatly extrapolated from known data, therefore not too reliable. Nearshore profiles could be of greater value if projected all the way to the toe of the bluff. This would give much needed information concerning width of beach and toe location. Topographic maps cannot adequately supply such information. Further field work and testing is required for a detailed engineering design.

Bathymetric coverage of Lake Erie is limited and dated. Such information is necessary to perform an adequate design analysis. More coverage at regular intervals would significantly increase the accuracy

of design wave calculations.

Prediction of longshore transport rate for Lake Erie is rather difficult. Computing the rate from historical changes in the topography of the littoral zone is long and complicated. An empirical method would simplify this calculation. The SPM (1975) has an empirical energy flux method, but it is not applicable to the Great Lakes environment.

The integrated protection design should be studied from a construction management viewpoint. Although a preliminary design has been suggested, a detailed engineering design requires that practical problems be considered such as equipment, availability and usage, transportation of materials and equipment, access to the work site, and construction sequence, to name just a few. Current economic evaluation should also be accomplished.

Demonstration programs are extremely useful in studying the effectiveness of structural protection measures. Through these programs, questionable methods can be studied and finally rejected or accepted. The results of such programs should be widely distributed and studied. The current testing of gabion structures in Michigan and at Geneva State Park should be closely monitored to determine whether gabions can be effective in the Great Lakes environment or not.

A detailed engineering design can be developed for an integrated protective system, but the real test of the integrated system is its implementation. In order to implement the design, a consensus must be

reached between persons of different social background, political affiliation, and economic status. A study should be made to determine if such a consensus is possible or not. Engineering designs are a waste of time and money if they cannot be implemented.

The private property owner does not sufficiently understand the mechanics of bluff recession to construct his own protective device. Several structures have failed due to increased back pressure, insufficient protection against scour, and overtopping. Additionally, some satisfactory structures are being flanked because there is no neighboring structure with which to connect. Some structures and actions, such as dumping fill, are temporary in nature. Individual efforts have not proven very satisfactory.

The preliminary investigation of this Lake County reach indicates that an integrated protective system can limit the shoreline recession. Physical factors remain constant and the bluff material appears to be homogeneous throughout the reach. Therefore, a detailed engineering design, to include complete soils' analysis, borings, groundwater tests, economic evaluations, etc., should be accomplished. This report and Stanley (1969) sufficiently cover this reach of Lake Erie shoreline that further general studies would be redundant. The effectiveness of an integrated protective system can be proven only if a design is implemented. A structure built in this location will facilitate such an effectiveness determination.

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APPENDIX A

SOILS TESTING

SAMPLE #1. SHORE CLUB OF WILLOWICK

Table A-1. Liquid Limit Determination (Weight in grams)

Tare	Wet Weight	Dry Weight	Tare Weight	Water Weight	Soil Weight	Percent Moisture (Y)	Blows (X)
5	33.24	31.93	27.13	1.31	4.80	27.29	37
22	35.14	33.03	25.63	2.11	7.40	28.51	21
33	26.03	23.62	15.22	2.41	8.40	28.69	19
341	26.13	23.92	16.22	2.21	7.70	28.70	16
406	28.83	26.43	18.12	2.40	8.31	28.88	13

Table A-2. Linear Regression Analysis of Liquid Limit Data
(Calculator Programmed)

Test Data			Best Fit Line Data		
Blows (X)	Log X	Percent Moisture (Y)	Blows (X)	Log X	Percent Moisture (Y)
13	1.114	28.88	15	1.176	28.85
16	1.204	28.70	20	1.301	28.40
19	1.279	28.69	25	1.398	28.05
21	1.322	28.51	30	1.477	27.77
37	1.568	27.29	35	1.544	27.53

LL = 28.1%

$$FI = \text{Slope} = \frac{Y_2 - Y_1}{\log X_2 - \log X_1} = -3.6$$

Correlation Coefficient = -.96

Table A-3. Plastic Limit Determination (Weight in grams)

Tare	Wet Weight	Dry Weight	Tare Weight	Water Weight	Soil Weight	Percent Moisture (PL)
A4	8.21	7.41	3.20	0.80	4.21	19.00
406	18.02	17.72	16.22	0.30	1.50	20.00

$$PL = \frac{(19.00 + 20.00)}{2} = 19.5$$

Table A-4. Natural Moisture Content Determination (Weight in grams)

Tare	Wet Weight	Dry Weight	Tare Weight	Water Weight	Soil Weight	Percent Moisture (NMC)
99	47.45	45.25	27.33	2.20	17.92	12.3

$$PI = LL - PL = 28.1 - 19.5 = 8.6$$

$$LI = \frac{NMC - PL}{PI} = \frac{12.3 - 19.5}{8.6} = -.84$$

SAMPLE #2. EASTLAKE SEWAGE TREATMENT PLANT

Table A-5. Liquid Limit Determination (Weight in grams)

Tare	Wet Weight	Dry Weight	Tare Weight	Water Weight	Soil Weight	Percent Moisture (Y)	Blows (X)
1	21.32	20.02	15.12	1.30	4.90	26.53	39
40	33.53	31.73	25.13	1.80	6.60	27.27	35
33	24.73	22.72	15.22	2.01	7.50	26.80	31
341	26.43	24.22	16.22	2.21	8.00	27.63	24
406	25.03	23.52	18.12	1.51	5.40	27.96	21
437	27.53	25.13	16.72	2.40	8.41	28.54	14

Table A-6. Linear Regression Analysis of Liquid Limit Data (Calculator Programmed)

Test Data			Best Fit Line Data		
Blows (X)	Log X	Percent Moisture (Y)	Blows (X)	LOG X	Percent Moisture (Y)
14	1.146	28.54	15	1.176	28.47
21	1.322	27.96	20	1.301	27.93
24	1.380	27.63	25	1.398	27.52
31	1.491	26.80	30	1.477	27.18
35	1.544	27.27	35	1.544	26.89
39	1.591	26.53			

LL = 27.5

$$\text{FI} = \text{Slope} = \frac{Y_2 - Y_1}{\log X_2 - \log X_1} = -4.3$$

Correlation Coefficient = -.95

Table A-7. Plastic Limit Determination (Weight in grams)

Tare	Wet Weight	Dry Weight	Tare Weight	Water Weight	Soil Weight	Percent Moisture (PL)
C7	8.51	7.81	3.80	0.70	4.01	17.46
40	37.74	36.94	33.14	0.80	3.80	21.05

$$\text{PL} = \frac{17.46 + 21.05}{2} = 19.3$$

Table A-8. Natural Moisture Content Determination

(Weight in Grams)

Tare	Wet Weight	Dry Weight	Tare Weight	Water Weight	Soil Weight	Percent Moisture (NMC)
63	46.75	44.45	27.43	2.30	17.02	13.5

$$PI = LL - PL = 27.5 - 19.3 = 8.2$$

$$LI = \frac{NMC - PL}{PI} = \frac{13.5 - 19.3}{8.2} = -.71$$

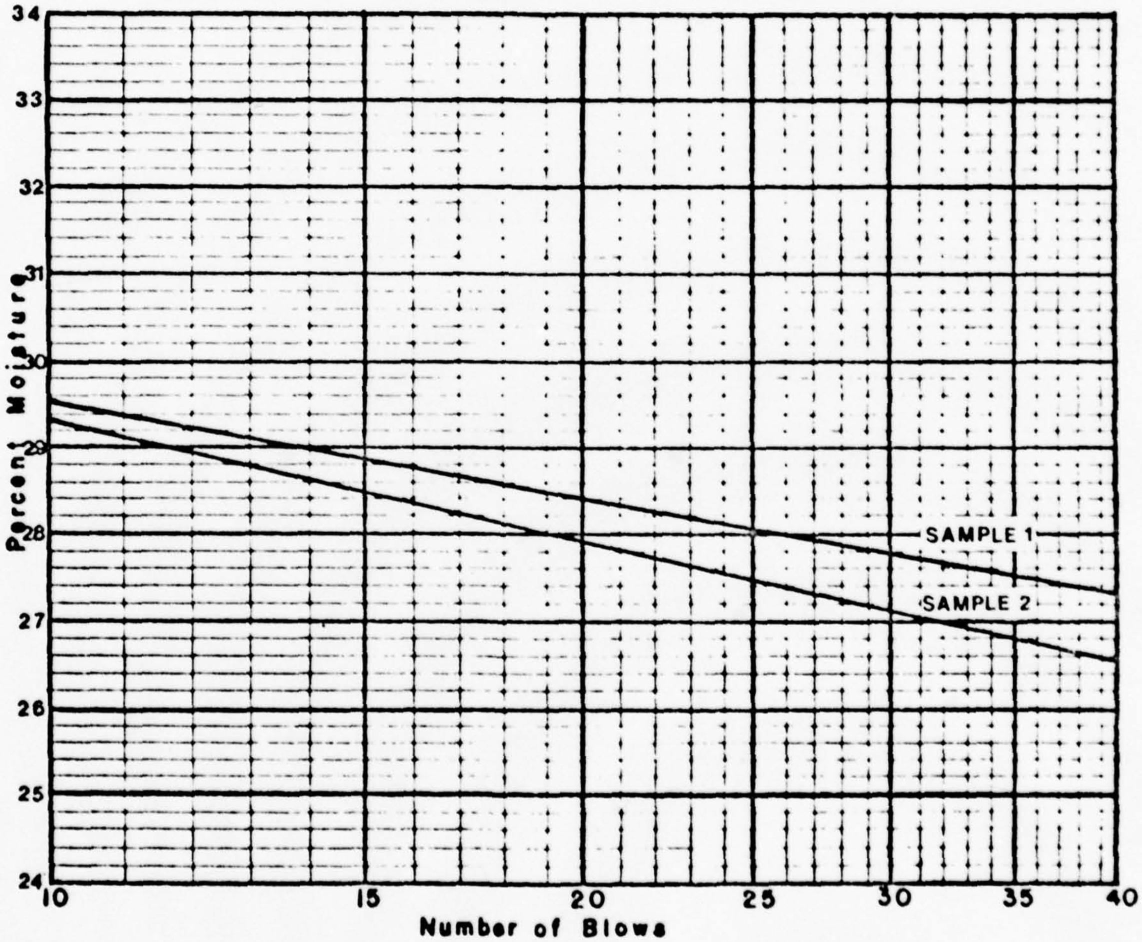


Figure A-1

Liquid Limit

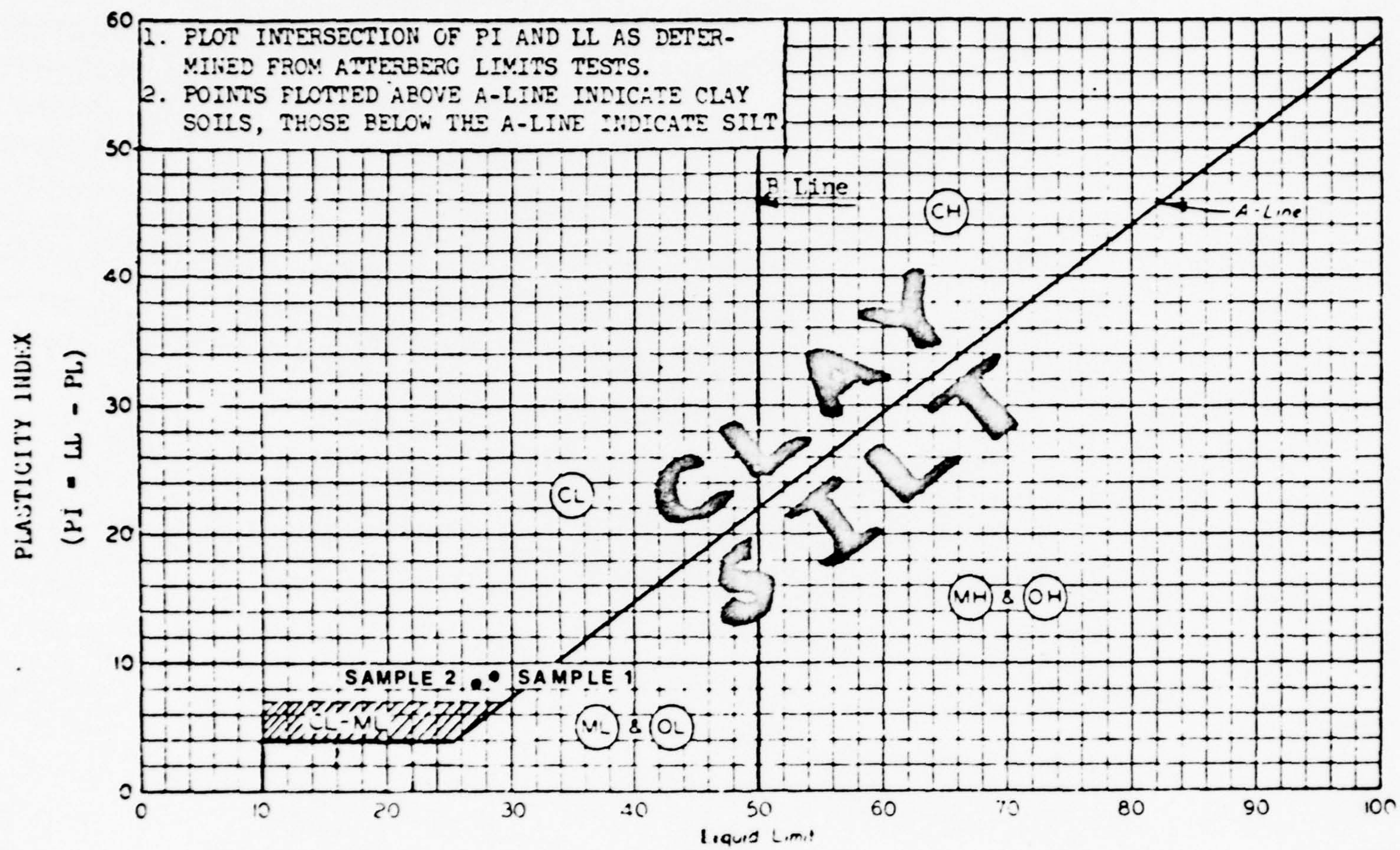


Figure A-2

Plasticity Chart

SIGNIFICANCE LEVEL TEST

Can the material in the reach be considered homogeneous based on two Atterberg Limits Test? The following significance test from Neter and Wasserman's book (1974) indicates the material is homogeneous at the 0.05 significance level.

Sample #1

$b_0 = 33.08$
$b_1 = -3.60$

$$SSE = \Sigma Y^2 - b_0 \Sigma Y - b_1 \Sigma XY$$

$SSE_1 = 4{,}038.425 - (33.08)(142.070) - (-3.60)(183.903)$

$SSE_1 = 0.799$

$n_1 - 2 = 5 - 2 = 3$

$\frac{0.799}{3} = 0.266$

Sample #2

$b_0 = 33.51$
$b_1 = -4.29$

$SSE_2 = 4525.444 - (33.51)(164.730) - (-4.29)(232.072)$

$SSE_2 = 0.932$

$n_2 - 2 = 6 - 2 = 4$

$\frac{0.932}{4} = 0.233$

$$F^* = \frac{0.266}{0.233} = 1.142$$

$\alpha = 0.10$ $\quad F(\alpha/2, n_1-1, n_2-1) = F(.05,3,4) = 6.59$ Table A-4

$$F(1-\alpha/2, n_1-1, n_2-1) = F(.95,3,4) = \frac{1}{F(.05,4,3)} = 0.110$$

$0.110 \leq 1.142 \leq 6.59$ therefore variances are equal.

Full Model

$SSE(F) = SSE_1 + SSE_2 = 0.799 + 0.932 = 1.731$

Reduced Model

$b_o = 34.03$

$b_1 = -4.52$

$SSE(R) = \Sigma Y^2 - b_o \Sigma Y - b_1 \Sigma XY$

$SSE(R) = 8563.869 - (34.03)(306.8) - (-4.52)(415.975)$

$SSE(R) = 3.672$

$$F^* = \frac{SSE(R) - SSE(F)}{(n_1+n_2-2)-(n_1+n_2-4)} \div \frac{SSE(F)}{(n_1+n_2-4)} = \frac{3.672-1.731}{(5+6-2)-(5+6-4)} \div \frac{1.731}{(5+6-4)} = 3.925$$

$\alpha = 0.05$ $F(.95,2,7) = 4.74$ Table A-4

If $F^* \leq F(.95,2,7)$ then same sample at 0.05 significance level.

$3.925 \leq 4.74$ therefore same sample.

APPENDIX B

LONGSHORE TRANSPORT CALCULATIONS

VOLUMETRIC CALCULATIONS

Figure B-1 shows the volumes to be calculated. Table B-1 contains the measurements from photogrammetric analysis.

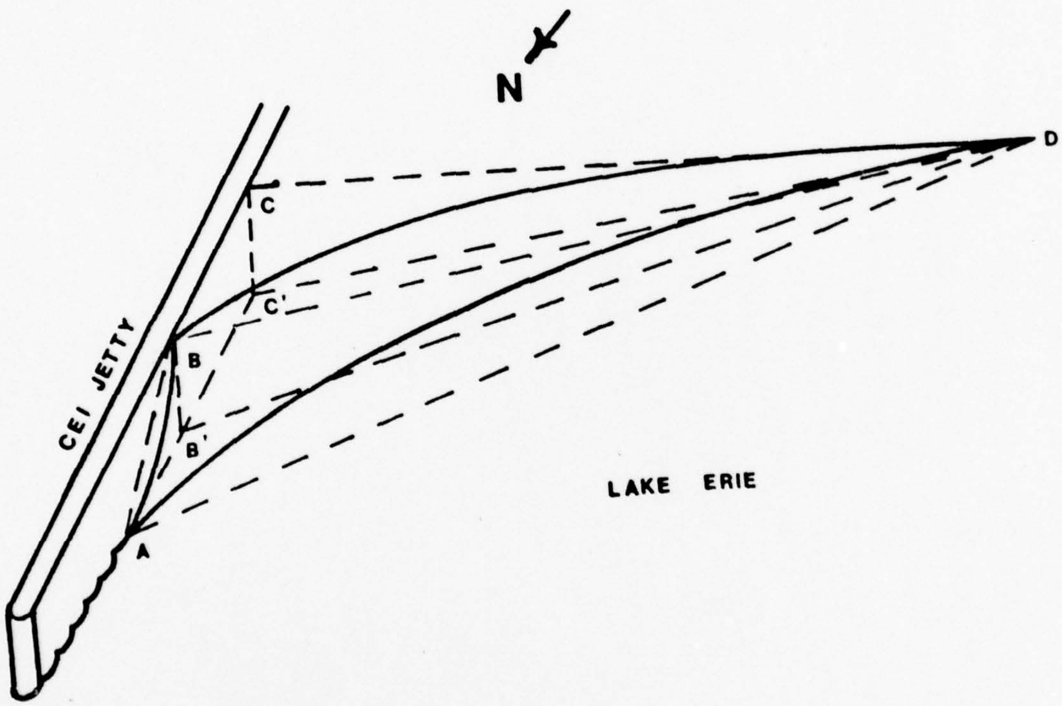


Figure B-1

Beach Fillet Volume

Table B-1. Volumetric Measurements

Year	Scale (ft/mm)	Elevation (ft)			Length (Measured in mm, Actual in ft)							
		A	B and C	D	AB'		BC and B'C'		CD		DA	
					Meas.	Act.	Meas.	Act.	Meas.	Act.	Meas.	Act.
1968	16.67	0	6.0	0	25.4	423.5	1.3	21.5	122.8	2047.0	126.2	2104.0
1974	16.27	0	5.5	0	0.7	11.5	21.5	350.0	123.5	2009.5	127.3	2071.0
1978	16.82	0	4.5	0	6.4	107.5	23.5	395.0	119.4	2008.5	123.0	2069.0

1968 PHOTOS

Volume (total) = Volume BCC'B'D + Volume ABB'D

Volume BCC'B'D = 1/3[(6)(21.5)](2,047) = 88,021 ft^3

Volume ABB'D = 1/3[1/2(3)(423.5)](2,047) = 433,452 ft^3

Volume (total) = 88,021 + 433,452 = 521,473 ft^3 = 57,941 yd^3

1974 PHOTOS

Volume (total) = Volume BCC'B'D + ABB'D

Volume BCC'B'D = 1/3[(6)(350)](2,009.5) = 1,406,650 ft^3

Volume ABB'D = 1/3[1/3(6)(12.5)](2,009.5) = 16,746 ft^3

Volume (Total) = 1,406,650 + 16,746 = 1,423,396 ft^3 = 158,155 yd^3

1978 PHOTOS

Volume (total) = Volume BCC'B'D + Volume ABB'D

Volume BCC'B'D = 1/3[(6)(395)](2,008.5) = 1,586,715 ft^3

Volume ABB'D = 1/3[1/3(6)(143)](2,008.5) = 191,477 ft^3

Volume (total) = 1,586,715 + 191,477 = 1,778,192 ft^3 = 197,577 yd^3

NET LONGSHORE TRANSPORT RATE

Qn (68-74) = (158,155 - 57,941)/6 = 16,702 yd^3/yr

Qn (74-78) = (197,577 - 158,155)/4 = 9,856 yd^3/yr

Qn (68-78) = (197,577 - 57,941)/10 = 13,964 yd^3/yr

Therefore, let Qn = 14,000 yd^3/yr since the longer period of change will tend to smooth out short term fluctuations.

GROSS LONGSHORE TRANSPORT RATE

With the percentage of wind duration from east (X_e) and west (X_w) known, an estimate of Qg can be made. Table 12 and Figure 8 percentages are both used in order to determine two estimates. The estimates are based on the fact that:

Qn = Q right - Q left = 14,000 yd^3/yr

Q right = 14,000 yd^3/yr + Q left

Qg = Q right + Q left = 14,000 yr^3/yr + 2 Q left

Wind (total) = Wind (onshore) + Wind (offshore)

Wind (onshore) = Wind (right) + Wind (left)

Table 12	Figure 8
Wind (right) = 37%	Wind (right) = 28.4%
Wind (left) = 19%	Wind (left) = 23.5%
Wind (perpendicular) = 14%	Wind (perpendicular) = 11.1%
100% Wind (onshore) = (X_e) Wind (right) + (X_w) Wind (left)	100% Wind (onshore) = (X_e) Wind (right) + (X_w) Wind (left)
$\frac{X_e}{100} = \frac{37}{56}$ $X_e = 66\%$ $\frac{X_w}{100} = \frac{19}{56}$	$\frac{X_e}{100} = \frac{28.4}{51.9}$ $X_e = 55\%$ $\frac{X_w}{100} = \frac{23.5}{51.9}$
$X_w = 34\%$	$X_w = 45\%$

Q right $= .66$ Qg   Q left $= .34$ Qg

$Qg = 14{,}000\ yd^3/yr + 2(.34Qg) =$

$43{,}750\ yd^3/yr$

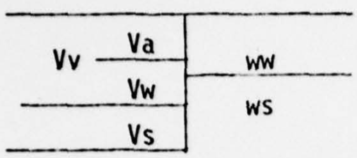
Q right $= .55$ Qg   Q left $= .45$ Qg

$Qg = 14{,}000\ yd^3/yr + 2(.45Qg) =$

$140{,}000\ yd^3/yr$

APPENDIX C

STABILITY CALCULATIONS



Let: $G_s = 2.70$
$w_w + w_s = 110$ lb
NMC = 13%
$V = 1\ ft^3$

$\frac{w_w}{w_s} = .13 \qquad w_w = .13\ w_s$

$.13\ w_s + w_s = 110$ lb $\qquad w_s = 97.3$ lb $\qquad w_w = 12.7$ lb

$V_s = w_s/2.70(62.4\ lb/ft^3) = 97.3\ lb/2.70(62.4\ lb/ft^3) = 0.58\ ft^3$

$V_w = w_w/(62.4\ lb/ft^3) = 12.7\ lb/(62.4\ lb/ft^3) = 0.20\ ft^3$

$V_a = 1 - 0.58 - 0.20 = 0.22\ ft^3$

$V_v = 1 - 0.58 = 0.42\ ft^3$

Moisture Content at PL = 19.4% | Moisture Content at LL = 28%

$w_w = (.194)(97.3\ lb) = 18.9$ lb | $w_w = (.28)(97.3\ lb) = 27.2$ lb

$w_w + w_s = 18.9 + 97.3 = 116.2$ lb | $w_w + w_s = 27.2 + 97.3 = 124.5$ lb

Weight change from PL to LL $= \frac{124.5\ lb - 116.2\ lb}{110\ lb} = 7.5\%$

APPENDIX D

WAVE CALCULATIONS

Table D-1. Wave Celerity Ratio at Contours

Parameters	d(ft)	d+5.1(ft)	d/Lo	tanh $2\pi d/L$	C_1/C_2	C_2/C_1
	6	11.1	.0396	.4780	1.21	.826
T=7.4 sec	12	17.1	.0611	.5798	1.13	.885
	18	23.1	.0825	.6574	1.09	.917
Lo=280 ft	24	29.1	.1039	.7197	1.07	.935
	30	35.1	.1254	.7708	1.05	.952
	36	41.1	.1468	.8127		
	6	11.1	.0356	.4553	1.21	.826
T=7.84 sec	12	17.1	.0548	.5529	1.14	.877
	18	23.1	.0740	.6289	1.10	.909
Lo=312 ft	24	29.1	.0933	.6905	1.07	.935
	30	35.1	.1125	.7414	1.06	.943
	36	41.1	.1317	.7838		
	6	11.1	.0338	.4445	1.22	.820
T=8.02 sec	12	17.1	.0521	.5408	1.14	.877
	18	23.1	.0704	.6159	1.10	.909
Lo=328 ft	24	29.1	.0887	.6769	1.08	.926
	30	35.1	.1070	.7277	1.06	.943
	36	41.1	.1253	.7706		

CONDITION: ANGLE #1 $T = 7.40$ sec $Lo = 280$ ft $Ho = 10.5$ ft

$$m = \frac{41.1}{6600} = \frac{1}{161} = .006$$

From refraction analysis: $\alpha_o = 29°$ $\alpha = 16°$ $b_o = 1.6$ cm $b = 1.7$ cm

$$\frac{Ho}{Lo} = .0375$$

Plate C-6 (SPM): KrKs = 0.94

Refracted and Shoaled Height - Hrs = (10.5)(0.94) = 9.87 ft

$$Kr = \left[\frac{\cos \alpha_o}{\cos \alpha}\right]^{1/2} = \left[\frac{\cos 29}{\cos 16}\right]^{1/2} = 0.95 \qquad Kr = \left[\frac{b_o}{b}\right]^{1/2} = \left[\frac{1.6}{1.7}\right]^{1/2} = 0.97$$

$$Kr = \frac{0.95 + 0.97}{2} = 0.96$$

Ho' = HoKr = (10.5 ft)(0.96) = 10.08 ft

Figure 7-3 (SPM): $\frac{Ho'}{T^2} = \frac{10.08 \text{ ft}}{(7.40 \text{ sec})^2} = .184$ use m = 0.020 (Conservative)

gives 1.06 = Hb/Ho', therefore Hb = (1.06)(10.08) = 10.68 ft

Hb > Hrs, therefore indicates unbroken wave of 9.87 ft striking the bluff. This is not reasonable based on experience, toe depth = 3 ft would mean dry beach between waves - impossible. Slope in Figure 7-3 cannot be extrapolated to solve this anomaly. Therefore, work backwards using Figure 7-4 and 7-5 to find out if wave breaks.

Figure 7-4 (SPM): $\frac{ds}{T^2} = \frac{3 \text{ ft}}{(7.4)^2} = .055$, m=0.0, $\frac{Hb}{ds} = .78$ therefore

Hb = .78(3) = 2.34 ft

Figure 7-5 (SPM): m=.02, $\frac{Hb}{T^2} = \frac{2.34}{(7.4)^2} = .043$ therefore Hb/Ho' = 1.75

$$Ho = \frac{Hb}{Kr(Hb/Ho')} = \frac{2.34}{.96(1.75)} = 1.39 \text{ ft}$$

Since Ho = 10.5 ft, it will in fact break, contrary to above indications.

Figure 7-2 (SPM): $\frac{Hb}{T^2} = \frac{2.34}{(7.4)^2} = 0.43$ use m=0 (conservative)

gives B curve - 1.28 = db/Hb = db/10.5 therefore db = 13.44 ft

α curve - 1.49 = db/Hb = db/10.5 therefore db = 15.65 ft

Structure will be subject to broken waves if ds < 13.44.

CONDITION: ANGLE #2 T = 7.84 sec Lo = 312 ft Ho = 11.8 ft

$$m = \frac{41.1}{5720} = \frac{1}{139} = .007$$

From refraction analysis: $\alpha_0 = 0$ $\alpha = 0$ $b_0 = 1.6$ cm $b = 1.6$ cm

Plate C-6 (SPM): KrKs >> 2.3

Refracted and Shoaled Height - H_{rs} >> (11.8 ft)(2) = 23 ft

$$Kr = \left[\frac{\cos \alpha_0}{\cos \alpha}\right]^{1/2} = 1 \qquad Kr = \left[\frac{b_0}{b}\right]^{1/2} = 1 \qquad Kr = 1$$

Ho' = HoKr = (11.8 ft)(1) = 11.8 ft

Figure 7-3 (SPM): $\frac{Ho'}{T^2} = \frac{11.8}{(7.84)^2} = .192$ Use m = 0.020 (conservative)

gives $\frac{Hb}{Ho'} = 1.05$ therefore Hb = (1.05)(11.8) = 12.39 ft

Hb < Hrs therefore breaking will occur before maximum height can be reached.

Figure 7-2 (SPM): $\frac{Hb}{T^2} = \frac{12.39 \text{ ft}}{(7.84s)^2} = .202$ use m = 0 (conservative)

B curve - 1.28 = db/Hb = db/12.39 therefore db = 15.86 ft

α curve - 1.54 = db/Hb = db/12.39 therefore db = 19.08 ft

Structure will be subject to broken waves if ds < 15.86 ft.

Assign ds = 3.0 ft (average condition)

Figure 7-4 (SPM): $\frac{ds}{T^2} = \frac{3}{(7.84)^2} = 0.49$ use m = 0 (conservative)

Gives Hb/ds = 0.78 therefore Hb = (0.78)(3) = 2.34 ft = design breaker height.

CONDITION: ANGLE #3 T = 8.02 sec Lo = 328 ft Ho = 10.8 ft

$m = \frac{41.1}{7490} = \frac{1}{180} = .006$

From refraction analysis: $\alpha_0 = 31°$ $\alpha = 11°$ $b_o = 1.6$ cm $b = 1.9$ cm

$\frac{Ho}{Lo} = .0329$

Plate C-6 (SPM): KrKs = 1.11

Refracted and Shoaled Height - H_{rs} = (10.8 ft)(1.11) = 11.99 ft

$Kr = \left[\frac{\cos \alpha_o}{\cos \alpha}\right]^{1/2} = \left[\frac{\cos 31}{\cos 11}\right]^{1/2} = 0.93$ $Kr = \left[\frac{b_o}{b}\right]^{1/2} = \left[\frac{1.6}{1.9}\right]^{1/2} = .92$

$Kr = \frac{0.93 + .92}{2} = .92$

Ho' = HoKr = (10.8 ft)(0.92) = 9.98 ft

Figure 7-3 (SPM): $\frac{Ho'}{T^2} = \frac{9.98}{(8.02)^2} = 0.155$ use m = 0.020 (conservative)

gives 1.1 = Hb/Ho' therefore Hb = (1.1)(9.98 ft) = 10.98 ft

Figure 7-2 (SPM): $\frac{Hb}{T^2} = \frac{10.98}{(8.02)^2} = 0.171$ use m = 0 (conservative)

B curve - 1.28 = db/Hb = db/11.04 therefore db = 14.13 ft

∝ curve - 1.52 = db/Hb = db/11.04 therefore db = 16.89 ft

Hb < Hrs therefore breaking will occur before maximum height can be reached. Structure will be subject to broken waves if ds < 14.13 ft.

Assign ds = 3.0 ft (average condition)

Figure 7-4 (SPM): $\frac{ds}{T^2} = \frac{3}{(8.02)^3} = 0.047$ use m = 0 (conservative)

gives 0.78 = Hb/ds therefore Hb = (0.78)(3 ft) = 2.34 ft = design breaker height.

APPENDIX E

STRUCTURE CALCULATIONS

GABION SEAWALL

H_b = 2.5 ft H_o' = 11.8 ft   T = 7.84 sec d_s = 3 ft (average)

$\cot\theta = \frac{1}{1.2} = 0.83$ $\frac{H_o'}{T^2} \approx \frac{11.8}{(7.84)^2} = 0.20$

Use arguments above to interpolate between Figures 7-8 and 7-9, SPM.

Figure 7-8 (SPM): $\frac{d_s}{H_o'} = 0$ $\frac{R}{H_o'} = 0.89$

Interpolated Value: $\frac{d_s}{H_o'} = \frac{3}{11.8} = 0.25$ $\frac{R}{H_o'} = 1.34$

Figure 7-9 (SPM): $\frac{d_s}{H_o'} = 0.45$ $\frac{R}{H_o'} = 1.7$

R = 1.34(2.5) = 3.35 ft (uncorrected for scale effects)

Figure 7-13 (SPM): $\tan\theta = \frac{1.2}{1} = 1.2$ (H = 1.5 ft - 4.5 ft approaches 1.17 asymptotically)

k = 1.17   Corrected R = 1.17(3.35) = 3.92 ft

Design uses gabion = riprap, therefore riprap correction required.

Compare smooth slope to riprap slope (1 to 1.5). Use arguments specified and interpolate.

$\cot\theta = \frac{1.5}{1} = 1.5$ $\frac{H_o'}{T^2} = 0.20$

Figure 7-8 (SPM): $\frac{d_s}{H_o'} = 0$ $\frac{R}{H_o'} = 0.87$

Interpolated value: $\frac{d_s}{H_o'} = 0.25$ $\frac{R}{H_o'} = 1.28$

Figure 7-9 (SPM): $\frac{ds}{Ho'} = 0.45$ $\frac{R}{Ho'} = 1.6$

Figure 7-15(riprap 1:1.15) using $\frac{ds}{Ho'} = 0.38$ (closest to actual value of 0.25)

$\frac{R.}{Ho'} = 1.05$

Reduction due to riprap = $\frac{1.05}{1.28} = 0.82$, Rriprap = 0.82(3.92) = 3.2 ft

Bluff protected to (Design lake level $_{hign}$) + Rriprap = 573.7 + 3.2 = 576.9 ft (IGLD)

GABION REVETMENT

Hb = 2.5 ft Ho' = 11.8 ft T = 7.84 sec ds = 3.5 ft

cot 26° = 2.05 $\frac{Ho'}{T^2} = \frac{11.8}{(7.84)^2} = 0.20$

Figure 7-8 (SPM): $\frac{ds}{Ho'} = 0$ $\frac{R}{Ho'} = 0.85$

Interpolated value: $\frac{ds}{Ho'} = \frac{3.5}{11.8} = 0.30$ $\frac{R}{Ho'} = 1.22$

Figure 7-9 (SPM): $\frac{ds}{Ho'} = 0.45$ $\frac{R}{Ho'} = 1.40$

R = 1.22(2.5) = 3.05 ft (uncorrected for scale effects)

Figure 7-13 (SPM): tan 26° = 0.49 (H = 1.5 ft to 4.5 ft) k = 1.149

Corrected R = 1.149(3.05) = 3.5 ft

$\cot\theta = \frac{1.5}{1} = 1.5$ $\frac{Ho'}{T^2} = 0.20$

Figure 7-8 (SPM): $\frac{ds}{Ho'} = 0$ $\frac{R}{Ho'} = 0.87$

Interpolated value: $\frac{ds}{Ho'} = \frac{3.5}{11.8} = 0.30$ $\frac{R}{Ho'} = 1.36$

Figure 7-9 (SPM): $\frac{ds}{Ho'} = 0.45$ $\frac{R}{Ho'} = 1.6$

Figure 7-15 (riprap 1:1.5) using $\frac{ds}{Ho'} = 0.38$ (closest to 0.30)

$\frac{R}{Ho'} = 1.05$

Reduction due to riprap $= \frac{1.05}{1.36} = .77$

Rriprap = (.77)(3.5) = 2.7 ft

Bluff protected to (Design lake level $_{high}$) + Rriprap =

573.7 + 2.7 = 576.4 ft (IGLD)

ROCK REVETMENT - RUNUP

Hb = 2.5 ft Ho' = 11.8 ft T = 7.84 sec ds = 3 ft (average)

cot 26 = 2 $\frac{Ho'}{T^2} = \frac{11.8}{(7.84)^2} = 0.20$

Figure 7-8 (SPM): $\frac{ds}{Ho'} = 0$ $\frac{R}{Ho'} = .85$

Interpolated value: $\frac{ds}{Ho'} = \frac{3}{11.8} = 0.25$ $\frac{R}{Ho'} = 1.17$

Figure 7-9 (SPM): $\frac{ds}{Ho'} = 0.45$ $\frac{R}{Ho'} = 1.43$

R = 1.17(2.5) = 2.9 ft (uncorrected for scale effects)

Figure 7-13 (SPM): tan 26 = .49 (H - 1.5 ft to 4.5 ft) k = 1.15

Corrected R = 1.15(2.9) = 3.34 ft

$\cot \theta = \frac{1.5}{1} = 1.5$ $\frac{Ho'}{T^2} = 0.20$

Figure 7-8 (SPM): $\frac{ds}{Ho'} = 0$ $\frac{R}{Ho'} = 0.87$

Interpolated value: $\frac{ds}{Ho'} = 0.25$ $\frac{R}{Ho'} = 1.28$

Figure 7-9 (SPM): $\frac{ds}{Ho'} = 0.45$ $\frac{R}{Ho'} = 1.6$

Figure 7-15 (riprap 1:1.5) using $\frac{ds}{Ho'} = 0.38$ (closest to 0.25) $\frac{R}{Ho'} = 1.05$

Reduction due to riprap = $\frac{1.05}{1.28} = 0.82$

Rriprap = 0.82(3.34) = 2.7 ft

Bluff protected to (Design lake level $_{high}$) + Rriprap =

573.7 + 2.7 = 576.4 ft (IGLD)

ROCK REVETMENT - ARMOR ROCK WEIGHT

$$W = \frac{wr\, H^3}{K_D \left(\frac{wr}{ww} - 1\right)^3 \cot\theta}$$ (Hudson's formula)

$wr = 155\ \mathrm{lb/ft}^3$ (average for concrete-actual dependent on where quarried)

$\theta = 26°$ $\qquad$ $K_D = 2.5$ (Table 7-6: rough angular, 2 layers, random)

$H = 2.5$ ft $\qquad$ $ww = 62.4\ \mathrm{lb/ft}^3$

$$W = \frac{(155)(2.5)^3}{2.5(155/62.4 - 1)^3 \cot 26} = 148\ \mathrm{lb}$$

Wmax = 2W = 296 lb

Table 7-1 (SPM): $k\Delta = 1.15$, $p = 37$ (quarrystone, rough, 2 layers *random*)

$h = 2$ $\qquad$ $W = 148$ lb $\qquad$ $wr = 155\ \mathrm{lb/ft}^3$

Table 7-11 (SPM): 300 lb quarrystone is 1.40 ft diameter

$A = 17\ \mathrm{ft} \times 12\ \mathrm{ft} = 204\ \mathrm{ft}^2$

$$Nr = Ank\Delta\left[1 - \frac{P}{100}\right]\left[\frac{wr}{W}\right]^{2/3} = (204)(2)(1.15)\left[1 - \frac{37}{100}\right]\left[\frac{155}{148}\right]^{2/3} = 305$$

$$r = nk\Delta\left[\frac{W}{wr}\right]^{1/3} = (2)(1.15)(296/155)^{1/3} = 3\ \mathrm{ft}$$

Per 12 ft beach front:

(305)(296 lb) = 90,280 lb = 45.1 ton